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EVALUATION OF STRAND TRANSFER AND DEVELOPMENT LENGTHS IN PRETENSIONED GIRDERS WITH SEMI-LIGHTWEIGHT CONCRETE

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Indiana
Department
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16. Abstract

Indiana has been using lightweight aggregate consisting mostly of expanded shale in the production of prestressed concrete bridge girders for several projects. The lightweight aggregate has been used as partial replacement for regular gravel or crushed limestone coarse aggregate. This semi-lightweight concrete weighs around 2080 kg/m^3.

In a study sponsored by the FHWA, the current AASHTO equations for the calculation of transfer and development lengths of prestressing strand were found to be unconservative in the case of lightweight concrete members with unit weight less than 1920 kg/m^3. This finding raises a question regarding the applicability of the same equations to semi-lightweight concrete members.

The objective of this study co-sponsored by the Indiana Department of Transportation and the Federal Highway Administration is to determine if the current AASHTO Specifications are applicable to semi-lightweight pretensioned concrete bridge members. The study addresses the transfer and development lengths of 13.3 mm and 15.2 mm pretensioned strand in semi-lightweight concrete girders with concrete compressive strength of 48 MPa for the 13.3 mm strand and 69 MPa for the 15.2 mm strand. Two strand types were considered in this study.

The results of the laboratory evaluation suggest that the current transfer length of 50 strand diameters is adequate for semi-lightweight concrete if longitudinal cracks are not present along the transfer length. In the presence of longitudinal splitting cracks, the measured transfer length increased to approximately 70 strand diameters. It is recommended that the current requirements for strand development length be enforced at a distance "d_p" from the point of maximum moment towards the free end of the strand. The quantity "d_p" is the distance from the extreme compression fiber to the centroid of the prestressing steel but not less than 80% of the overall member height. In the specimens tested, web-shear cracks, intercepting the transfer length near the member ends, were not present.

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

Figure Page

IMPLEMENTATION SUGGESTIONS

The following recommendations are made based on the results of this study, and for the strand types and concrete mixes evaluated.

1. The current assumption for transfer length of 50 strand diameters was conservative in the absence of longitudinal splitting cracks at the member ends. In the one specimen where longitudinal splitting was noted, the transfer length measured was 70 strand diameters. Therefore, an estimate for transfer length of 50 strand-diameters can be used when checking shear provisions for prestressed members with semi-lightweight concrete in the absence of longitudinal splitting cracks. Otherwise, it is recommended to use an estimate for transfer length of 70 strand diameters.

2. A shift in the location of the critical section may occur due to flexure-shear cracking. Thus, it is suggested that the current requirements for development length be enforced at a section located a distance "d_p" from the critical section based on flexural requirements in the direction of its free. In this check, d_p is the distance from the extreme compression fiber to the centroid of the prestressed reinforcement, but no less than 80% of the overall member height. This recommendation may appear to be too conservative at first glance. However, for shallow members, checking development-length requirements at a small distance of d_p will not be overtaxing on design. For larger members with fully bonded strands, the issue of development length is seldom, a critical factor in the design. It must be noted, that all the multiple strand specimens were designed to avoid web-shear cracking near the member ends.

The presence of a shear crack, intercepting the transfer length of the strand being developed at the member end, could result in the strands slipping prematurely.

CHAPTER 1 - INTRODUCTION

1.1 Introduction and Background

The utilization of new materials in structures often leads to savings in construction costs and sometimes also to improved structural performance. In recent years, the availability of higher concrete strengths has reduced expenses by increasing the maximum span lengths that can be bridged using standard girder cross sections. However, with longer span lengths the self-weight of the prestressed sections has become an increasingly larger portion of the total design load for the bridge. Therefore, in order to reduce the dead load of the concrete girder, lightweight aggregate is often employed.

In the state of Indiana lightweight aggregate consisting mostly of expanded shale has been used to produce semi-lightweight concrete prestressed girders in the bridge projects listed in Table 1. In these projects the semi-lightweight concrete used weighs around 2080 kg/m^3 (130 pcf) compared to 2320 kg/m^3 (145 pcf) in normal weight concrete. The semi-lightweight concrete is obtained by partially replacing the gravel or limestone coarse aggregate with the lightweight one.

1.2 Problem Statement

In a recent study sponsored by the FHWA [1], the applicability of the current AASHTO [2] equations for the evaluation of transfer and development lengths of prestressing strands was evaluated in pretensioned light-weight concrete beams. The unit weight of the concrete was less than 1920 kg/m^3 (120 pcf) indicating that the coarse aggregate had been replaced in full with lightweight one. In this study, the current equations were found to be unconservative in estimating transfer and development lengths. In light of these findings, the applicability of the same equations to semi-lightweight concrete was also questionable, and no test data was available to provide an answer regarding the adequacy of the current AASHTO LRFD transfer and development length equations in the case of semi-lightweight concrete. Therefore, this research was co-sponsored by the Indiana Department of Transportation (INDOT) and Federal Highway Administration (FHWA) to determine the transfer and development lengths of prestressed strand when semi-lightweight concrete is used. This is necessary in order to determine the adequacy of existing structures and to provide recommendations for the design future projects using semi-lightweight concrete.

Table 1.1 Semi-Lightweight Concrete Bridge Projects in Indiana (2080 kg/m^3)

Contract No. Letting Date	Structure No.	Span Lengths (m)	f'c Girder (MPa)	f'c Slab (MPa)	Prestressing Type
R-21747 4/18/95	1-02-7488	2@35.05	48	31	Pretensioned/ Post-tensioned
B-21347 2/14/95	421-08-7399	39.95,2@40.46, 39.95	48	31	Pretensioned/ Post-tensioned
R--20961 1/10/95	231-79-7530	34.14,34.60, 34.37	48	35	Pretensioned/ Post-tensioned
B-21135 2/8/94	1-69-114-7564	38.56,42.67	48	31	Pretensioned/ Post-tensioned
B-20996 2/8/94	27-02-7487&J	38.56,40.08	48	31	Pretensioned/ Post-tensioned
B-20834 8/17/93	231-79-7531	53.09,10@53.49, 53.09	48	31	Pretensioned/ Post-tensioned
B-20396 8/17/93	Bartholomew 10725	32.51,2@32.77, 32.51	48	35	Pretensioned/ Post-tensioned
B-20525 1/12/93	Bartholomew 10724	32.16,13@32.46, 32.16	48	35	Pretensioned/ Post-tensioned
B-19711 10/24/91	231-28-2571	32.00,10@40.69	48	31	Pretensioned/ Post-tensioned

1.3 Scope

Since all the semi-lightweight prestressed girder bridges built in Indiana prior to this study had a concrete design strength of 48 MPa (7000 psi), the majority of this study focused on transfer and development length determination in members with a similar concrete design mix containing 13.3 mm (1/2"–special) diameter prestressing steel. However, in anticipation of higher strength concrete in the future, tests were also conducted on semi-lightweight girders having a target compressive strength of 69 MPa (10,000 psi) and containing 15.2 mm (0.6") diameter prestressing steel. Chapter 2 of this report describes the experimental program in detail, while Chapter 3 presents the experimental results obtained. The conclusions drawn from this study, along with recommendations for implementing the findings in design, are presented in Chapter 4.

CHAPTER 2 - EXPERIMENTAL PROGRAM

2.1 Strand Validation (Moustafa Method)

In a Special Report by Logan [3] published in the March-April 1997 edition of the PCI Journal, it was concluded that there is a significant difference in bond performance in pretensioned concrete beams among strands produced by different strand manufacturers. The report recommended that all 12.7 mm (0.5 in) diameter strand used in pretensioned applications be required to have a minimum average pull-out capacity of 160 kN (36 kips), with a standard deviation of 10% for a six-sample group, when embedded 460 mm (18 inches) in concrete test blocks. This test procedure has become known as the Moustafa method, named after Saad Moustafa who conducted pullout tests on similar specimens in the 1970's [4]. The Moustafa method is described in detail in Appendix-A.

The first task of this study, therefore, involved the fabrication and testing of similar pullout specimens to determine if the strand used in this study would meet the minimum average pullout capacity recommended by Logan. After numerous consultations with Don Logan [3], the transverse reinforcement used in the pullout specimens in this study was modified from that shown in Appendix A to provide a transverse tie next to each strand (See Figure 2.1-1). Since 1/2"-Special strand has a nominal diameter of 13.3 mm (0.522 in) instead of 12.7 mm (0.5 in), the corresponding minimum average pullout capacity for the 1/2"-Special strand (assuming a similar average bond stress at ultimate load) is 167 kN (37.6 kips). Using similar reasoning, the minimum average pullout capacity for 15.2 mm (0.6-in) strand is 192 kN (43.2 kips).

The prestressed concrete producer that supplied the girders for all of the semi-lightweight (SLW) girder bridges listed in Table 1.1 is CSR Hydro-Conduit, Lafayette, IN. During the last ten years Hydro-Conduit has used strand primarily from two suppliers, namely Florida Wire & Cable and Insteel. Therefore, at the outset of this experimental program, it was decided that test specimens would be fabricated using prestressing steel from both Florida Wire Cable and Insteel. References to strand supplied from these companies will be denoted by (FWC) and (IST), respectively.

As described in Appendix A, the Moustafa pullout test is based on a standard normal-weight (NW) concrete mix design. However, since this study is concerned with the bond between prestressing steel and SLW concrete, additional pullout specimens were fabricated which incorporated SLW concrete as well. One pullout specimen, having a target compressive strength of 69 MPa (10,000 psi), also contained 15.2 mm (0.6-in) strands. Table 2.1 shows the characteristics of each of the 4 pullout specimens tested in this study. Figures 2.1-2 shows the reinforcement used to fabricate a pullout specimen, while Figure 2.1-3 shows a typical pullout specimen prior to testing.

Table 2.1 Pullout Specimen Parameters

Specimen	Target Concrete Strength	Concrete Type	Number of Strands	Strand Producer	Strand Size
1	*Moustafa Mix	NW.	9 9	IST FWC	13.3 mm (1/2"- Special) 13.3 mm (1/2"- Special)
2	48 MPa (7000 psi)	NW	9 9	IST FWC	13.3 mm (1/2"- Special) 13.3 mm (1/2"- Special)
3	48 MPa (7000 psi)	SLW	9 9	IST FWC	13.3 mm (1/2"- Special) 13.3 mm (1/2"- Special)
4	69 MPa (10,000 psi)	SLW	9 9	IST IST	13.3 mm (1/2"- Special) 15.2 mm (0.6")

* See Appendix A for Moustafa Mix details.

2.2 Surface Condition Assessment

Many observers have noted differences in appearance, color, and residue of strand from different manufacturers. Therefore, attempts were made to document the initial surface condition of the strand used in this study. Visual appearance of the strand, in terms of color and signs of weathering, were noted for the strand used in the pullout and beam specimens. In addition, every piece of strand used in the pullout specimens was wiped with a white paper towel prior to tying into the rebar cage to remove residue and aid in the visual assessment of the initial surface condition. This process was also performed by Logan [3] prior to casting his pullout specimens and is described in Appendix A.

2.3 The Importance and Use of Transfer Lengths and Development Lengths

The "transfer length" is defined as the distance required to transfer the fully effective prestress force in the strand to the concrete. The transfer length is not a quantity specified in either the ACI [5] or AASHTO [2] codes. However, both codes suggest a transfer length of 50 strand diameters when checking shear provisions. The ACI Commentary to the Building Code (Section 12.9) provides a formula for calculating the transfer length that is based on the expression for development length. According to this formula, the transfer length (L_t) is given by:

$$L_t = \frac{f_{se}}{3} \times d_b \qquad (2\text{-}1)$$

where f_{se} is the effective stress (ksi) in the strand after all losses, and d_b is the nominal diameter of the strand in inches.

The "development length" is the bond length required to anchor the strand as it resists external loads on the member [6]. As external loads are applied to a flexural member, the member resists the increased moment demand through increased internal tensile and compressive forces. The increased tension in the strand is achieved through additional anchorage to the surrounding concrete. Thus, the development length is equal to the length required to transfer the effective prestress force (transfer length) plus an additional length required to develop the increase in strand tension produced by the external load demand. This additional length required to develop the maximum stress in the strand is often referred to as the "flexural bond length." The development length is specified by both the ACI and AASHTO Codes as:

$$L_d = \left(f_{ps} - \frac{2}{3} f_{se} \right) d_b \qquad (2\text{-}2)$$

where f_{ps} is the stress in the prestressed strand at nominal strength of the member (in ksi), f_{se}

and d_b are the same as in Equation 2-1.

The ACI Commentary assumptions for transfer and development of stress in prestressing strand are graphed in Figure 2.3-1 (Figure R12.9 in the Commentary). In this figure, the stress in the strand is plotted against the distance from the free end of the strand. The transfer length is represented by the first portion of the curve having a larger slope, while the flexural bond length is represented by the second portion of the curve.

Transfer lengths affect structural design considerations in two ways. First, as mentioned in the preceding paragraph, current code provisions for shear design of prestressed members are based on the amount of pre-compression in the member. Since the effective prestress varies approximately linearly from zero at the end of the member to the fully effective at the end of the transfer zone, significant deviations in the transfer length from the code-suggested 50-strand-diameters could mean inadequate performance of the member in shear.

The transfer length can also have a significant impact on the flexural behavior of prestressed members. Russell and Burns [6] found that anchorage failures were likely when flexural cracking of a beam propagated through the transfer zone of a pretensioned strand. Beams with de-bonded strand are especially susceptible to this phenomenon. Therefore, the value of the transfer length is important to determine whether flexural cracks will likely propagate into this zone prior to the member reaching nominal capacity.

Development-length requirements are typically "checked," rather than designed for. When a prestressed member is designed, required longitudinal reinforcement quantities are based on service-load stresses as well as calculations of nominal capacities. The ACI and

AASHTO codes prescribe reinforcement-ratio limits to ensure that ductility is provided through ample yielding of the prestressed reinforcement at ultimate loads. Thus, for flexural considerations, the designer calculates a nominal moment capacity of the prestressed section by estimating a final level of stress that will be achieved by the strand (f_{ps}). Based on the estimate of f_{ps}, the designer calculates a development length (L_d) by Equation 2-2. A check is then made to ensure that the strand will have a large enough embedment length (L_e) in the concrete to obtain the estimated stress at nominal capacity (f_{ps}).

The embedment length is defined as the bonded length of the prestressed strand from the beginning of bond to the critical section. In most design applications, and in the literature, the critical section is interpreted as the point of maximum moment [6]. ACI section 10.2.2 states that "critical sections for development of reinforcement in flexural members are at points of maximum stress and at points within the span where adjacent reinforcement terminates." Both the ACI and AASHTO codes imply that if the embedment length is greater than the development length ($L_e > L_d$) then the beam will be able to reach the nominal moment capacity and will fail in flexure. Conversely, if the embedment length is less than the development length ($L_e < L_d$) then bond failure will occur prior to the beam reaching its nominal capacity and the design is unsatisfactory. However, research has shown that bond failures may still occur when ($L_e > L_d$) if web shear cracking occurs and propagates into the transfer zone. Russell and Burns [6] recommended design procedures which take this into consideration when normal-weight concrete is used.

While considerable research has been published on the experimental determination of transfer and development lengths in members utilizing normal-weight concrete, with emphasis on structural behavior and implications for design, similar work for members made of semi-lightweight concrete is essentially absent from the literature. Therefore, the objective of this experimental program was to determine the transfer and development length of prestressed strand in semi-lightweight girders, and to assess the adequacy of current code provisions for the design of such members.

2.4 Transfer-length Measurements

Transfer lengths were experimentally determined by measuring concrete surface strains at the ends of test specimens. Stainless-steel points were secured to the specimens at 50 mm (2 in) spacings prior to detensioning the strands. The points were mounted using a 5-minute epoxy and were located at the depth of the strand. Distances were measured between points using a Whittemore gage that had a 254 mm (10 in) gage length and had a differential reading capability of 0.00254 mm (1/1000 in), with a perceived accuracy of twice this amount. Therefore, the resolution of the gage was about $20\mu\varepsilon$.

Surface strain readings were taken prior to detensioning, immediately after detensioning, and periodically during the first month after stripping. Transfer-length measurements were initially taken for the single-strand beam specimens discussed in Section 2.7. However, the value of surface strains in these beams from pre-compression (P/A) was relatively small (in the order of $200\ \mu\varepsilon$). This factor, coupled with the effects of rapid temperature changes in late Fall and moisture effects on the epoxy used to mount the points, often resulted in unreliable readings during these initial attempt.

Therefore, two additional specimens were fabricated specifically for measuring transfer lengths. These specimens had a cross-section that measured 100 mm x 150 mm (4 in x 6 in) and contained two concentric strands. One of the specimens contained strand that was produced by Florida Wire & Cable while the other specimen contained strand that was produced by Insteel. Whittemore readings were taken on both sides and both ends of each specimen, which was approximately 2400 mm (7'-10 1/2") long. These specimens proved to be superior for transfer-length measurements, as initial compressive strains were on the order of 3-4 times larger than those for the single-strand specimens. Also, the fabrication of these specimens occurred in late Spring, when conditions were more favorable for instrumentation. Figures 2.4-1 shows the formwork and reinforcement for the transfer-length specimens. Figure 2.4-2 shows the Whittemore points mounted on one of the transfer-length specimens.

2.5 Evaluation of Development Lengths

As indicated by the title of this section, development lengths are evaluated, rather than determined in experimental programs. This is typically done by designing test specimens which are loaded in such a way that the maximum moment occurs at the point in the beam where the provided embedment length L_e is equal to the calculated development length L_d. This point is commonly referred to as the "critical section."

Development-length testing in this experimental program consisted of testing nine single-strand specimens and six multiple-strand specimens. The test specimens in this study had fully bonded strands and were tested by applying loads from a hydraulic actuator that was located at a distance L_d from the end of the specimen. Loads were applied incrementally until failure of the members occurred. Interpretation of the test results is straight-forward. A flexural failure indicates that the embedment length is adequate to develop the strand, while a bond failure indicates that the embedment length is not sufficient and that the actual development length is larger than the calculated value.

2.6 Calculation of Development Lengths for Test Specimens

The ACI development length equation (Equation 2-2) was presented in Section 2.3. This equation considers the development length to be a function of 3 variables, namely

f_{se}, the effective stress in the strand after all losses (ksi),

d_b , the nominal diameter of the strand in inches, and

f_{ps} , the stress in the strand at nominal strength of the member (ksi).

Thus, the code-prescribed development length is not a single value that can be evaluated for a given strand. Instead, it is a function of both the strand properties and the properties of the member in which it is cast. Interestingly, for a given strand size and member, the development length may be calculated to be different values by different designers, depending on the assumptions which are made in calculating f_{se} and f_{ps}. From Equation 2-2 it can be seen that the calculated development length is largest when f_{ps} is maximized and f_{se} is minimized. In other words, if the designer overestimates f_{ps} while underestimating f_{se} (by overestimating prestress losses), then the calculated development length will be "longest." While there may be other implications on design (i.e. member sizing, stress and camber calculations, etc.), the result of calculating excessively long development lengths (in terms of actual bond performance) is conservative since it will result in longer required embedment lengths.

However, the converse may not be true. If the designer underestimates f_{ps} while at the same time overestimating f_{se} (by underestimating prestress losses), then the calculated development length will be minimized. This calculation will, in turn, lead to shorter required embedment lengths for the strand and bond failure could occur prior to the member reaching nominal capacity.

Since this research was aimed at evaluating the validity of the current code equations for development lengths when semi-lightweight concrete is used, it was determined that, conservatively, the worst-case situations should be tested. With this in mind, the experimental tests in this study were designed so that the "shortest" development lengths that might realistically be calculated by designers would be tested. As discussed above, the "shortest" development lengths are calculated when f_{ps} is minimized and f_{se} is maximized. The stress in the strand at nominal strength of the member, f_{ps}, is typically estimated by either direct calculation from code equations or by a strain compatibility analysis. While the strain compatibility analysis is generally considered to be more accurate, especially when more than one layer of steel is provided, the code equations are typically more conservative and yield a lower estimate of f_{ps}.

Using the ACI Code [5], the stress in the strand at member nominal capacity may be estimated by the equations in section 18.7.2 of the code. For members with bonded prestressing tendons and no compression reinforcement, the formula for f_{ps} reduces to

$$f_{ps} = f_{pu}\left\{1 - \frac{\gamma_p}{\beta_1}\left[\rho_p \frac{f_{pu}}{f_c'}\right]\right\} \quad (2\text{-}3)$$

where f_{pu} = the specified tensile strength of the prestressed tendons in psi.

γ_p = a factor for the type of prestressing tendon used (= 0.28 for low-lax strand).

β_1 = a factor used to enable ultimate flexural capacity calculations to be made by representing the concrete in compression by an equivalent rectangular stress block.

ρ_p = the ratio of prestressed reinforcement = A_{ps}/bd_p, where A_{ps} is the area of the prestressed reinforcement in the tension zone, b is the width of the compression face of the member, and d_p is the distance from the extreme compression fiber to the centroid of the prestressed reinforcement.

f_c' = the specified compressive strength of the concrete in psi.

Thus, in order to test the most severe condition, the stress in the strand at nominal capacity of the test specimens was calculated using Equation (2-3). In addition, the effective prestress after all losses, f_{se}, was calculated by assuming only 8% total losses. This value was also used by Logan [3] and is a practical minimum value that might be calculated by design engineers.

2.7 Single-Strand Development-Length Specimens

Nine single-strand development-length specimens were fabricated and tested in this study. The purpose of the single-strand specimens was to provide an inexpensive means to conduct multiple development-length tests with the same concrete and strand supplier combinations. Table 2.2 shows the concrete and strand parameters of the single-strand test beams. The single-strand beams were used for two development-length tests each – one per end. Shear reinforcement was provided only in the central portion of the beams to ensure that this region would be intact after the first end of the beam was tested. Note, the central portion of the beam was part of the loaded span for the testing of both ends of the single-strand beams.

The shear reinforcement in the center region of the beams was purposefully not centered in the beam. This was to provide the ability to increase the embedment length at one end of the test specimens in the event that initial beams tests based on the calculated development lengths would experience bond failure. However, as will be further discussed in Chapter 3, all single-strand beams were tested at the calculated development lengths of 1870 mm and 2170 mm.

The nomenclature used for the single-strand specimens is the following:

"[Concrete Strength & Type]-[Strand Type]-[Beam # within Series][Test End]"

Thus, the name "7SLW-IST-2L" would refer to a test specimen utilizing 7000 psi Semi-LightWeight concrete that had a single 13.3 mm (1/2"-Special) strand produced by InSTeel, and was the test at the "Long" end of specimen number 2 (of 3). The "long" end refers to the beam end with the greatest distance to the shear reinforcement located in the central region. Although all of the development-length beams in this study contained semi-lightweight concrete, the study was conducted concurrently with another study that utilized normal-weight concrete. Therefore, the term "SLW" was (unnecessarily) used in the naming of all specimens in this study. All of the 48 MPa single-strand beams used 13.3 mm (1/2"-Special) strand (produced by both Insteel and Florida Wire & Cable) while the 69 MPa single-strand beams used 15.2 mm (0.6") strand produced by Insteel only. Thus, the "Strand Type" portion of the nomenclature distinguishes between IST and FWC for the 48 MPa beams while using [0.6"] for the 69 MPa (10 ksi) beams.

Table 2.2 Single-Strand Semi-Lightweight Beam Parameters

Number of Beams	Strand Producer	Embedment Length	Concrete Strength	Strand Size
3	IST	1870 mm (6'-1 1/2")	48 MPa (7000 psi)	13.3 mm (1/2"- Special)
3	FWC	1870 mm (6'-1 1/2")	48 MPa (7000 psi)	13.3 mm (1/2"- Special)
3	IST	2170 mm (7'-1 1/2")	69 MPa (10,000 psi)	15.2 mm (0.6")

The single-strand specimens had a rectangular cross-section measuring 200 mm x 305 mm (8 in x 12 in), and contained a single prestressing strand located at a depth "d" of 255 mm (10 in) (see Figure 2.7-1). The width of 200 mm was slightly larger than the 165 mm (6 1/2") width used by Logan [3] for his single-strand specimens, in order to minimize the depth of concrete in compression and maximize the strain in the prestressed strand at the ultimate flexural capacity of the specimens. The increased specimen width was needed because both 13.3 mm (1/2 in - Special) and 15.2 mm (0.6 in) nominal diameter strands were tested in this study, whereas Logan tested specimens containing only (1/2 in) nominal diameter strands. Using the 200 mm width, the strain in the strands at nominal flexural capacity of the test beams was estimated at 2.7% based on a strain-compatibility analysis. Although this value was lower than the 3.5% value recommended by Buckner [7] for minimum strand strains in development-length specimens, it was larger than the 2.0% value calculated by Logan for his single-strand beams that failed by strand rupture.

Figures 2.7-2 and 2.7-3 show the loading arrangements corresponding to the 48 and 69 MPa (7000 and 10,000 psi) beams, respectively. As seen in these figures, the calculated development lengths (and thus the tested embedment lengths) for these beams were 1870 mm (6'-1 1/2") for the 48 MPa (7000 psi) beams and 2170 mm (7'-1 1/2") for the 69 MPa

(10,000 psi) beams. Loads were incrementally applied to the beams using a hydraulic ram powered by an electronically controlled power unit supplied by Nachi America. Values of load, and deflection at the applied load, and strand slip at the beam end were recorded throughout the entire loading sequence of all 18 tests (2 tests per beam for 9 beams). Figures 2.7-4 and 2.7-5 show test setup and strand slip measuring device used for the single-strand beams.

2.8 Multiple-Strand Development-Length Specimens

As previously mentioned, the purpose of the single-strand specimens in this study was to provide an inexpensive means to conduct numerous development-length tests on beams having the same concrete and strand supplier combination. However, the condition of a beam reinforced with only one strand is never encountered in actual bridge design. Therefore, the possible negative effect of multiple strands (at close spacing) on development lengths also needed to be addressed. Therefore, at the outset of this experimental program, it was decided that four full-scale specimens containing multiple strands would be tested in addition to the nine single-strand specimens. It was envisioned that these specimens would be designed based on the analysis of test data from the single-strand rectangular specimens.

As will be discussed in Chapter 3, results from tests on single-strand specimens indicated that the current ACI and AASHTO equations were appropriate for use with semi-lightweight concrete. Therefore, the multiple strand specimens were designed with an embedment length based on the current code provisions. These specimens each contained 5 strands and had a T-shaped cross-section with a depth of 535 mm (21 in) and a compression flange of 915 mm (36 in) (see Figure 2.8-1).

Three of the T-beam specimens contained 13.3 mm (1/2"-Special) diameter strands (two with strand manufactured by Insteel and one with strand manufactured by Florida Wire & Cable), and had a design compressive strength of 48 MPa (7000 psi). These three specimens were cast at the same time and in the same prestressing bed (See Figure 2.8-2). This was enabled by splicing all five strands in the span between the bulkheads of the beams using Insteel and Florida Wire & Cable strands (See Figure 2.8-3). Splicing of the strands ensured that all beams had the same initial tension in the strands. These beams will be referred to in this report as T-Beam IST1, T-Beam IST2, and T-Beam FWC. The fourth T-beam specimen (which will be referred to in this report as the "10 ksi T-Beam") contained 15.2 mm (0.6") diameter strands and had a target compressive strength of 69 MPa (10,000 psi). Table 2.3 contains the design parameters of the multiple-stranded T-beams.

Table 2.3 Multiple-Strand Semi-Lightweight T-Beam Parameters

Number of Beams Tested	Strand Producer	Embedment Length	Concrete Strength	Strand Size
2	IST	1870 mm (6'-1 1/2")	48 MPa (7000 psi)	13.3 mm (1/2"- Special)
1	FWC	1870 mm (6'-1 1/2")	48 MPa (7000 psi)	13.3 mm (1/2"- Special)
1	IST	2145 mm (7'-0 1/2")	69 MPa (10,000 psi)	15.2 mm (0.6")

Unlike the single-strand specimens, which each were tested at both ends, the multiple-strand T-beams were designed with a length that was approximately equal to twice the calculated development length, so that a point load applied at mid-span would effectively test the anchorage at both ends simultaneously. The actual length of the T-beams was 150 mm (6 in) longer than twice the calculated development length, because the load was applied to the beam through a 150 mm (6 in) wide steel plate. This ensured that the length of embedment of the strand (from the free end of the beam to the edge of the loading plate) coincided with the development length calculated based on the principles discussed in the Section 2.6. Figure 2.8-4 shows the dimensions and loading arrangement for the multiple-stranded T-Beams.

Design of shear steel using the ACI code provisions showed that 13 mm (#4) stirrups at 305 mm (15 in) spacings would provide sufficient shear reinforcement for all T-beams in this study (See Figure 2.8-5). However, the transverse reinforcement provided in the T-beams was 13 mm (#4) stirrups at 150 mm (6 in) spacings, or more than twice the code-required amount. Figure 2.8-6 shows the typical stirrup detailing for the T-beams. For ease of fabrication, and in keeping with current detailing practices for box-beams in Indiana, the stirrup assembly was fabricated and placed on top of the strands after they been tensioned (See Figure 2-8.7).

The T-beams were tested in the Karl H. Kettelhut Structural Engineering Laboratory at Purdue University. Loads were incrementally applied to the beams through a 150 mm wide x 610 mm long (6 in x 24 in) steel plate using a 978 kN (220-kip) capacity MTS hydraulic actuator. Values of load, mid-span deflection, and strand slip for all (5) strands both ends of the beam were recorded during the testing of each T-beam. Figures 2.8-8 and 2.8-9 show test setup and strand slip measuring device used for the multi-stranded T-beams.

2.9 The Importance of Stirrup Spacing on Longitudinal Steel Stress

The results of the development-length tests will be presented in Chapter 3 of this report. However, some of the findings are mentioned at this point because they led to the fabrication and testing of three additional T-beam specimens.

T-Beam FWC experienced bond failure prior to reaching the nominal moment capacity. A careful review of a videotaped recording of this load test in slow motion showed that an inclined flexure-shear crack occurred immediately prior to the strand slip and subsequent web-shear cracking. It is common knowledge that the initiation of inclined cracks in simply-supported beams will cause an increase in the tension demand closer to the support. This is easily seen when using a "truss model" to idealize the behavior of concrete beams. The ACI code accounts for this tension shift in flexural members with non-prestressed reinforcement (ACI 12.10.2) by requiring that longitudinal bars in tension be extended for a distance equal to the effective depth of the member beyond the point where they are required to resist flexure.

The behavior of T-Beam FWC observed by the principal investigators suggested that this shift in tension demand could not be "developed" in the prestressed reinforcement and resulted in bond failure. This behavior also suggests that the "critical section" for prestressed members referred to in ACI 12.9.1 may not correspond to the location of the maximum moment, but rather a point at some distance closer to the end of the beam. This idea is explained in the following paragraphs.

Figure 2.9-1 shows an idealized bilinear representation of the stress capacity in bonded prestressed tendons verses the distance from the free end of the strand (refer to Figure 2.3-1). For the T-beams, the development-length tests were conducted so that the maximum stress in the strands (f_{ps}) was produced at a distance from the end of the beam that was equal to the

ACI prescribed development length L_d. In this figure, the stress in the prestressed strand must lie below the bilinear curve at all locations in order for bond requirements to be satisfied. Note that at a distance (x) closer to the free end from the development length (L_d), the maximum permissible stress in the strand is equal to $(f_{ps}-\Delta f)$.

Figure 2.9-2a (from MacGregor [8]) shows the internal forces in a cracked beam without stirrups. For wide cracks, the aggregate interlocking force V_a disappears, along with V_d, V'_{cz} and C_1', and $T_2 = T_1$. In other words, the inclined crack has made the tensile force at point C a function of the moment at section A-B-D-E [8], and point C can now (arguably) be deemed the "critical section". For a beam without stirrups, having its nominal moment capacity (and corresponding stress f_{ps}) demanded at a distance (L_d) from the free end of the strand, the onset of flexure-shear cracking will produce a strand stress (at the crack location) that lies above the bilinear curve in Figure 2.9.1. If the ACI expression for strand development length represents the actual length required to develop the strand stress (f_{ps}) corresponding to nominal capacity for the beam then inclined cracking may lead to bond failure.

Figure 2.9-2b (also from MacGregor [8]) shows the internal forces in a cracked beam with stirrups. In this case, the presence of stirrups will ensure that there will always be a compression force C_1' and a shear force V'_{cz} acting on the part of the beam below the crack, and therefore T_2 will be less than T_1. However, even though the tension force at point C will be less than the tension at section A-B-D-E, the strand stress may still lie above the bilinear curve in Figure 2.9.1 and failure by bond can still occur. Figure 2.9-1 illustrates that a change in strand stress of (Δf) equal to (x/d_b) ksi must occur over the distance "x" from the point of maximum moment so that the strand stress (at distance "x" from the maximum moment) will not lie above the bilinear curve. Therefore, when an inclined crack occurs, extending from the point of maximum moment to a point "x" closer to the free end of the strand, a reduction in strand stress of (x/d_b) ksi must occur over the horizontal projection of the crack to preclude bond failure. This can be accomplished by providing extra transverse reinforcement across the crack.

The amount of required transverse reinforcement to cause the appropriate reduction in strand stress may be estimated using the model in Figure 2.9-3. This model, which assumes the inclined crack can be represented by a linear crack, is the basis for the calculation of required transverse reinforcement in Figure 2.9-4. The calculations in Figure 2.9-4 indicate that at least three 13 mm (#4) stirrups must cross a transverse crack in order to cause the appropriate reduction in strand stress and preclude possible bond failure. A post-failure inspection of T-Beam FWC indicated that only 1 or 2 stirrups crossed the inclined crack that preceded failure.

The model in Figure 2.9-3 is believed to give an upper-bound estimate of the transverse reinforcement required to reduce strand stress by the amount Δf shown in Figure2.9-1 because it (conservatively) ignores the contributions of dowel action by the longitudinal and transverse reinforcement. A lower-bound estimate of the required transverse reinforcement is shown in Figure 2.9-5, which uses the shear-friction approach of ACI Section 11-7. This calculation underestimates the required transverse reinforcement since it does not take into account the net tension occurring across the inclined crack. Using the shear-friction approach, only 110 to 150 mm^2 (0.41 to 0.48 in^2) of vertical reinforcement is required to cross the flexure-shear crack. Thus, it is believed that at least two, and possibly three 13 mm (#4) stirrups would need to cross an inclined flexure-shear crack in the 48 MPa (7 ksi) T-beams in order to reduce the strand stress by the amount suggested in Figure 2.9-1.

2.10 Additional (3) T-Beams

Based on the failure by bond of T-Beam FWC, and the reasoning discussed in Section 2.9, the investigators decided to fabricate and test three additional 48 MPa (7 ksi) T-Beam specimens. These T-Beams were identical in length, cross-section, and longitudinal reinforcement to the three T-Beams in Section 2.8, but all three utilized 13.3 mm (1/2”-Special) strand produced by Florida Wire & Cable. These additional three T-beams differed from the original three T-beams in the detailing and amount of the transverse reinforcement. Figure 2.10-1 shows the detailing of the transverse reinforcement used in the additional three T-beams. A two-piece stirrup assembly was used to allow the vertical stirrups to easily be placed below the strands prior to tensioning, thereby assuring they will be properly anchored according to ACI 12.13.2.1.

As indicated in the preceding paragraph, the amount of transverse reinforcement in the central portion of the beams was also varied for the additional three specimens. One T-Beam had 13 mm (#4) stirrups at 152 mm (6 in) on center throughout the entire beam. This beam was similar to T-Beam FWC that failed by bond and was used to determine if the transverse reinforcement detailing (stirrups anchored below the strand vs. stirrups resting on top of the strands) would effect ultimate performance. A second T-beam had 13 mm (#4) stirrups at 75 mm (3 in) on center in the middle portion and was used to test the hypothesis explained in

Section 2.9, namely that increased stirrup spacing can control the tension shift that supposedly resulted in bond failure of T-Beam FWC. The third additional T-Beam had 13 mm (#4) stirrups at 375 mm (15 in) on center in the middle portion of the beam. This corresponded to the ACI Code-required minimum amount (see Figure 2.8-5) of transverse reinforcement. The vertical stirrup detailing for the additional T-Beams is shown in Figure 2.10-2. Figure 2.10-3 is a photo of vertical stirrup assemblies similar to the ones used in the additional T-beams.

The reinforcement in the flanges of the additional T-Beams was similar to that in the original three T-beams discussed in Section 2.8. Figure 2.10-4 shows the detailing of the "top" flange assemblies used in each of the additional T-Beams. Figure 2.10-5 is a photo of the top flange reinforcement used in these beams. Table 2.4 lists the parameters of the additional three T-beams.

Table 2.4 Parameters of Additional (3) T-Beams

T-Beam Name	Strand Producer	Embedment Length	Concrete Strength	Strand Size
7SLW-FWC-3"	FWC	1870 mm (6'-1 1/2")	48 MPa (7000 psi)	13.3 mm (1/2"- Special)
7SLW-FWC-6"	FWC	1870 mm (6'-1 1/2")	48 MPa (7000 psi)	13.3 mm (1/2"- Special)
7SLW-FWC-15"	FWC	1870 mm (6'-1 1/2")	48 MPa (7000 psi)	13.3 mm (1/2"- Special)

It should be noted at this time that the 69 MPa (10 ksi) T-beam (which was cast after the additional three T-beams) also incorporated stirrup detailing similar to that for the additional T-beams. Stirrups were anchored below the strands for this beam (see Figure 2.10-1) and had a spacing in the middle portion of the beam of 75 mm (3 in), similar to one of the additional T-beams. Flange reinforcement for the 69 MPa T-beam was similar to that shown in Figure 2.10-3.

Table 2.4 Parameters of Additional (3) T-Beams

T-Beam Name	Strand Producer	Embedment Length	Concrete Strength	Strand Size
7SLW-FWC-3"	FWC	1870 mm (6'-1 1/2")	48 MPa (7000 psi)	13.3 mm (1/2"- Special)
7SLW-FWC-6"	FWC	1870 mm (6'-1 1/2")	48 MPa (7000 psi)	13.3 mm (1/2"- Special)
7SLW-FWC-15"	FWC	1870 mm (6'-1 1/2")	48 MPa (7000 psi)	13.3 mm (1/2"- Special)

It should be noted at this time that the 69 MPa (10 ksi) T-beam (which was cast after the additional three T-beams) also incorporated stirrup detailing similar to that for the additional T-beams. Stirrups were anchored below the strands for this beam (see Figure 2.10-1) and had a spacing in the middle portion of the beam of 75 mm (3 in), similar to one of the additional T-beams. Flange reinforcement for the 69 MPa T-beam was similar to that shown in Figure 2.10-3.

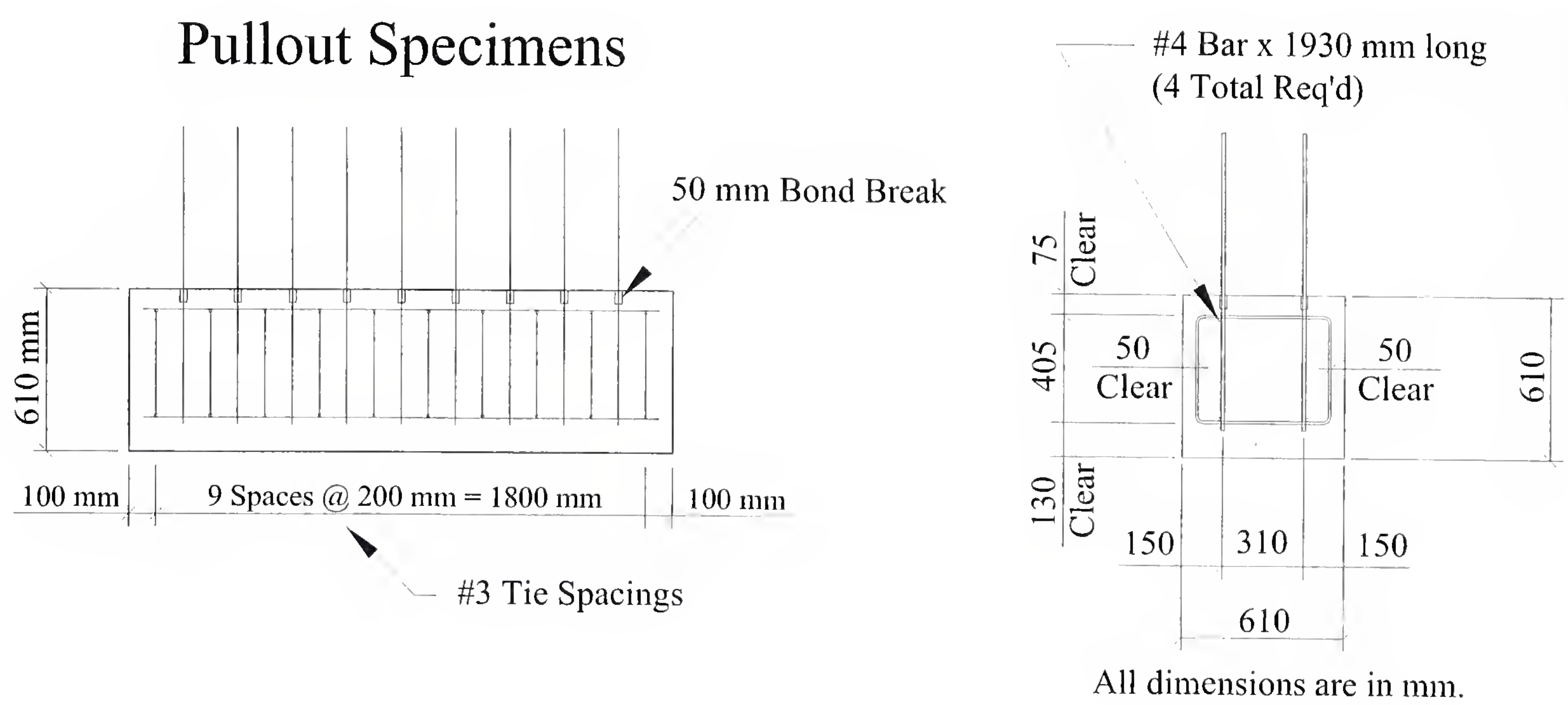


Figure 2.1-1 Details of the Moustafa Pullout Specimens used in this study.

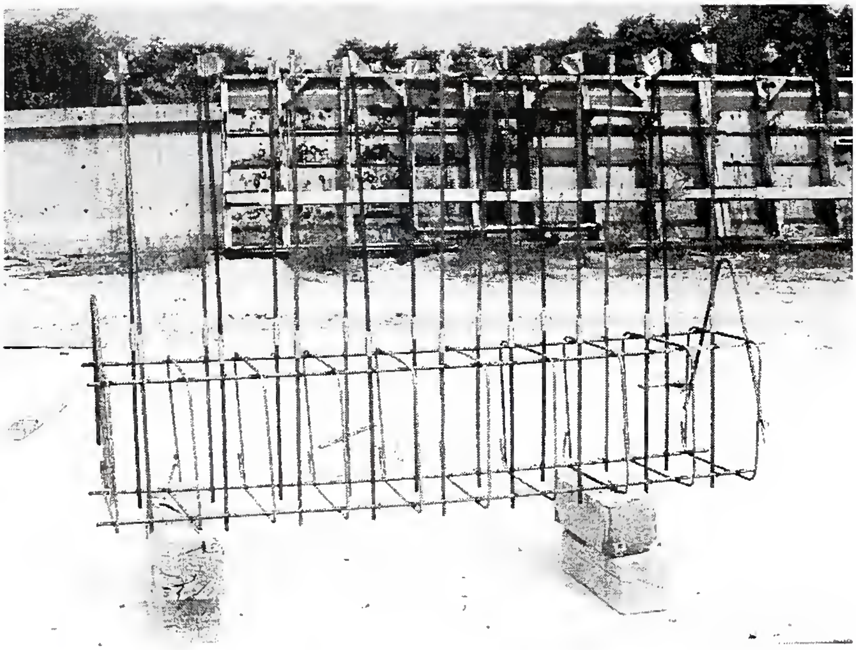


Figure 2.1-2 Reinforcement cage used in Moustafa pullout specimens.

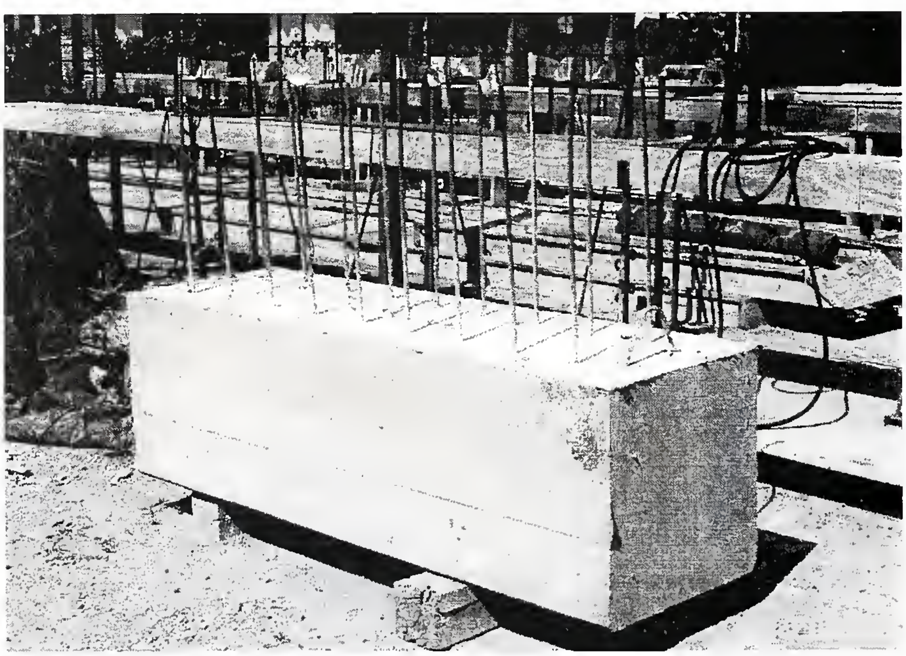


Figure 2.1-3 Typical Moustafa pullout specimen.

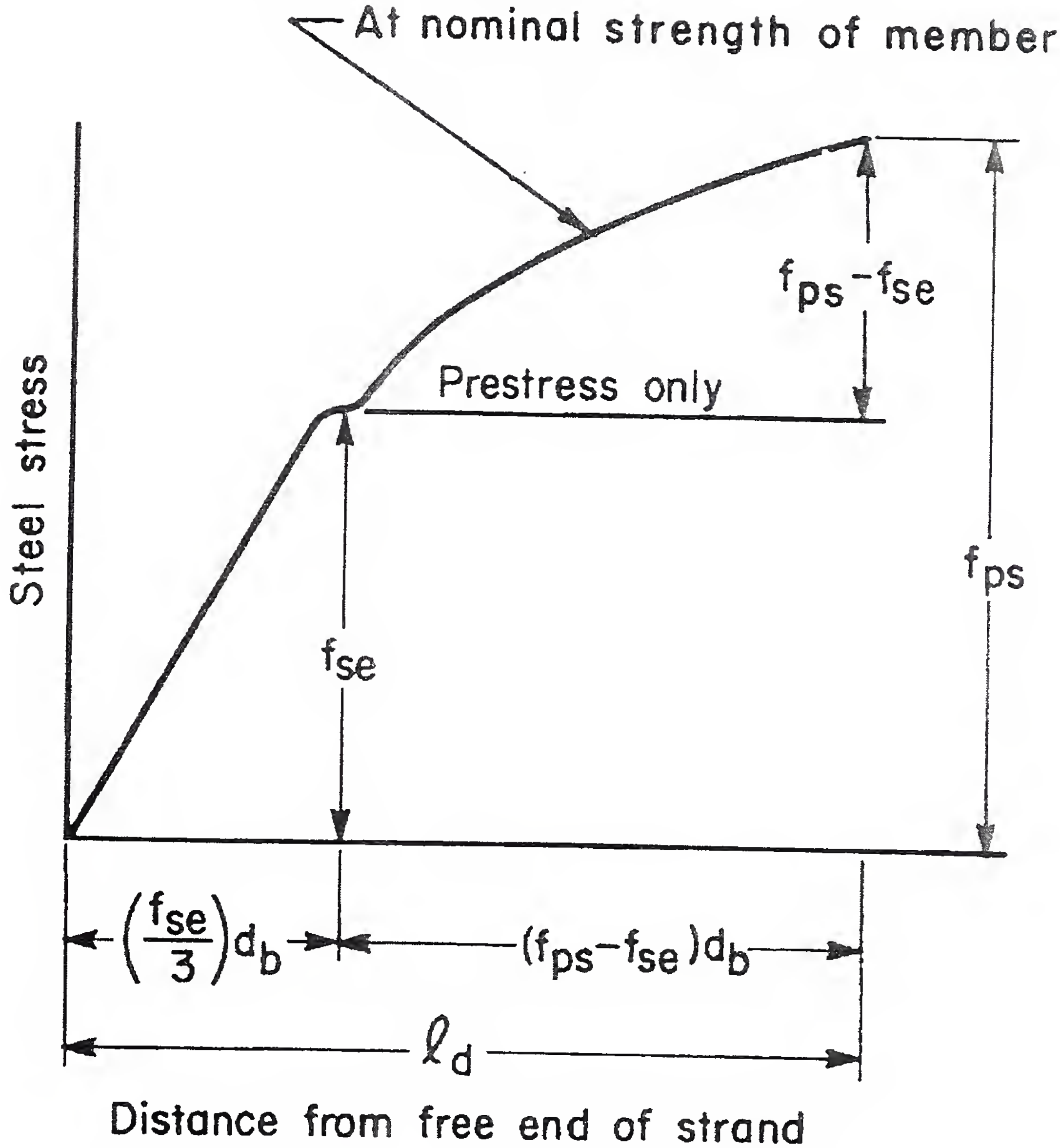


Figure 2.3-1 ACI Commentary Figure (R12.9) depicting transfer and development lengths.

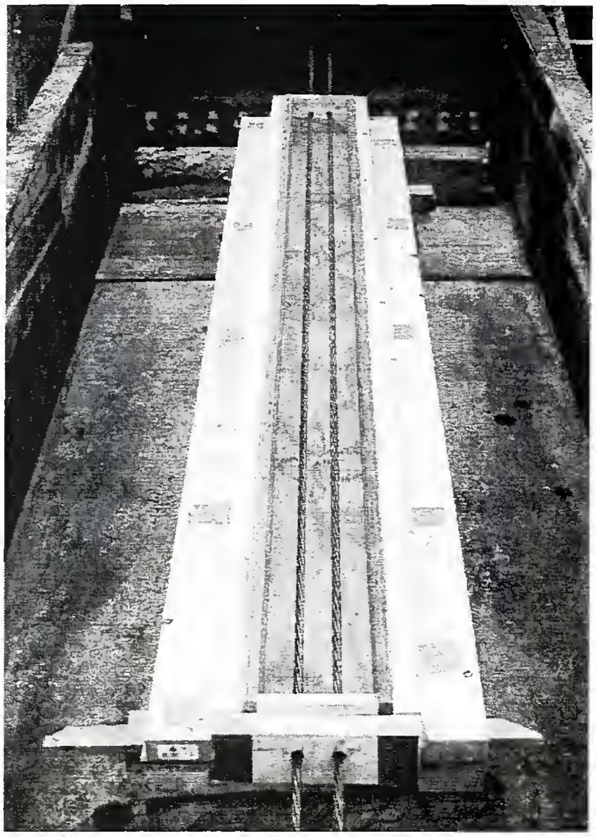


Figure 2.4-1 Photo of Transfer Length Specimen Formwork.

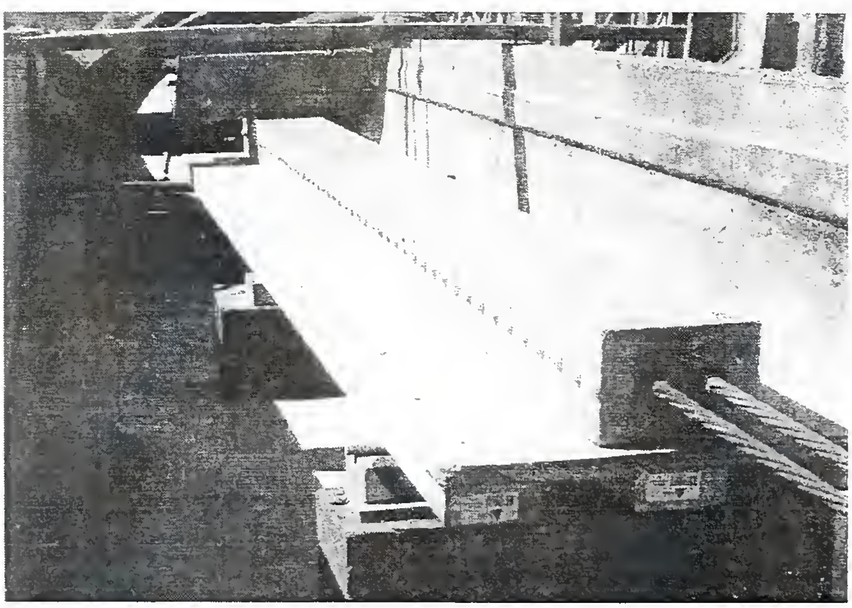


Figure 2.4-2 Photo showing Whittemore points mounted on transfer length specimen.

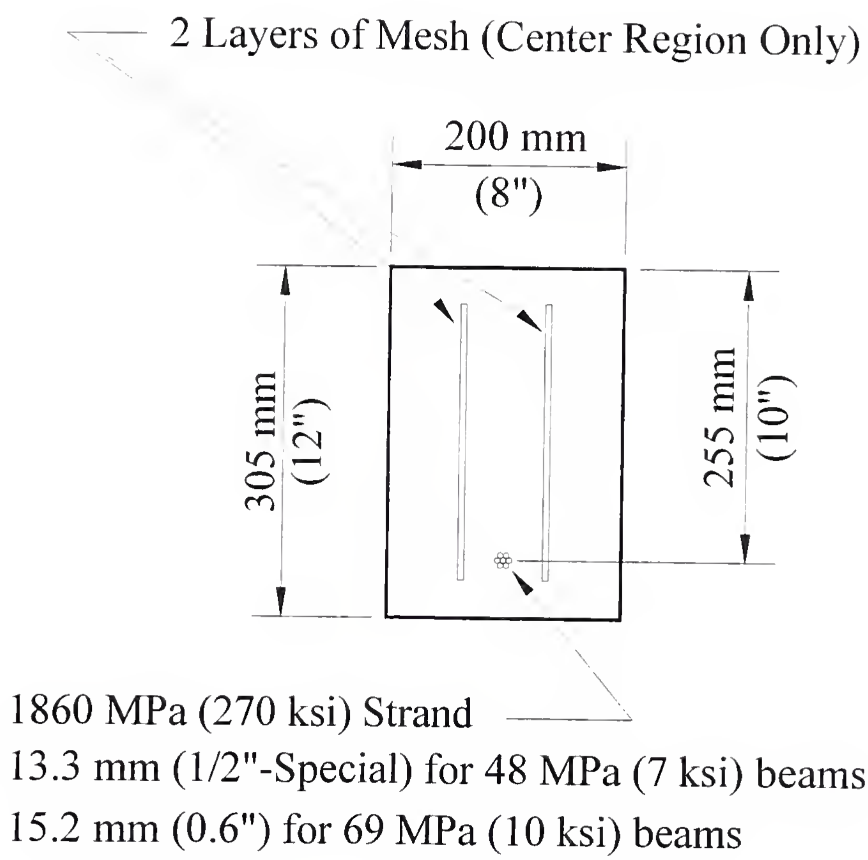


Figure 2.7-1 Cross-section of single-strand development length beams.

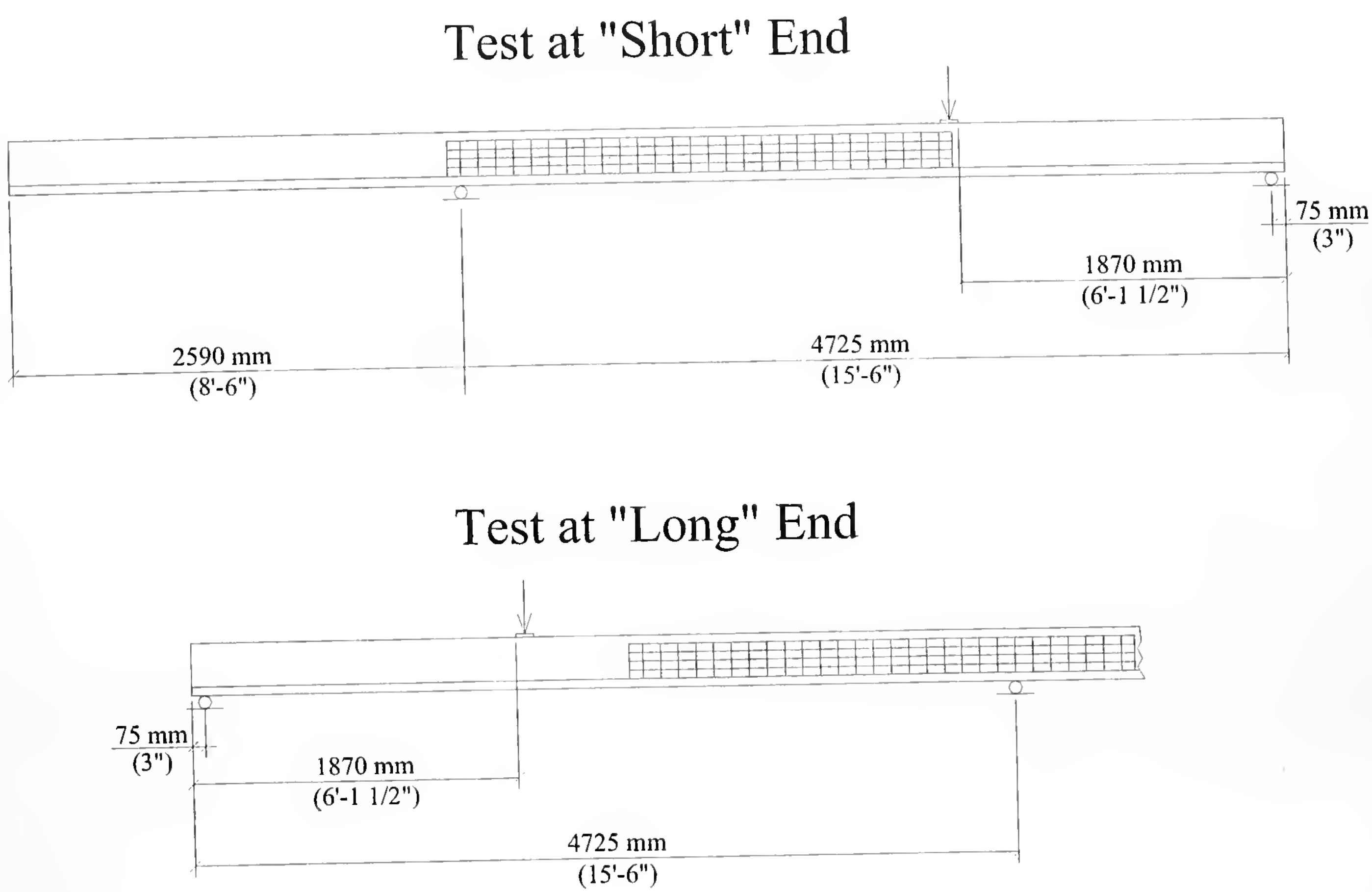


Figure 2.7-2 Loading arrangement for the 48 MPa (7000 psi) single-strand beams.

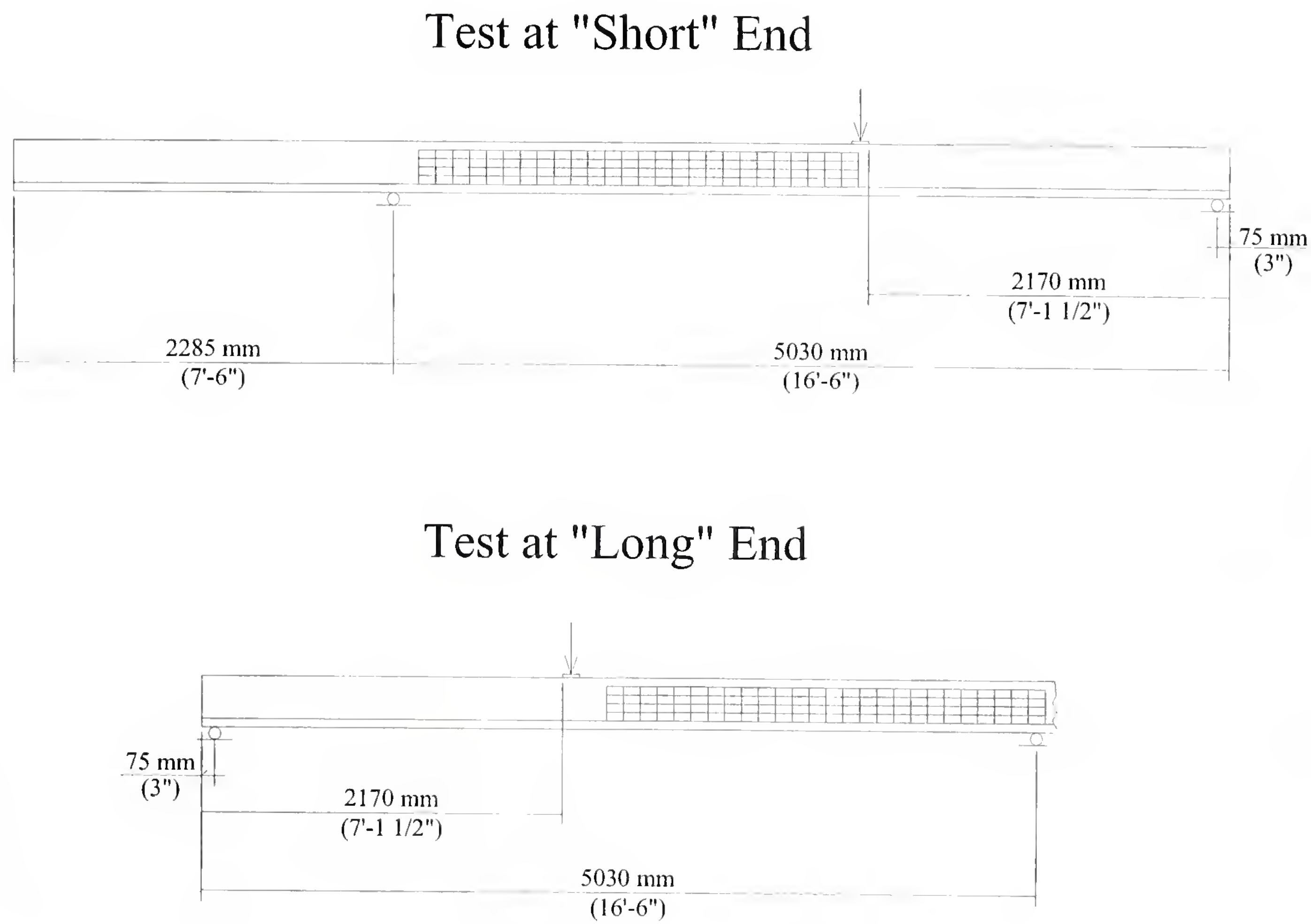


Figure 2.7-3 Loading arrangement for the 69 MPa (10,000 psi) single-strand beams.

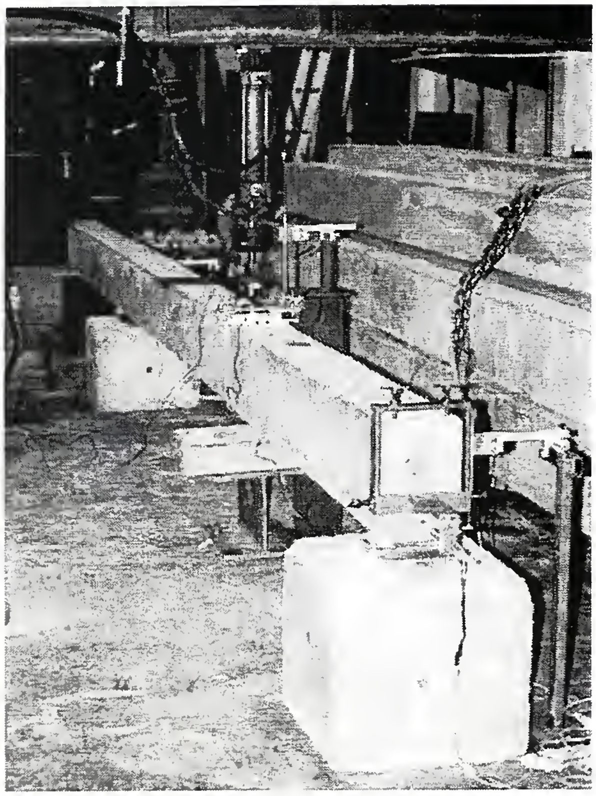


Figure 2.7-4 Photo showing test setup for single-strand beams.

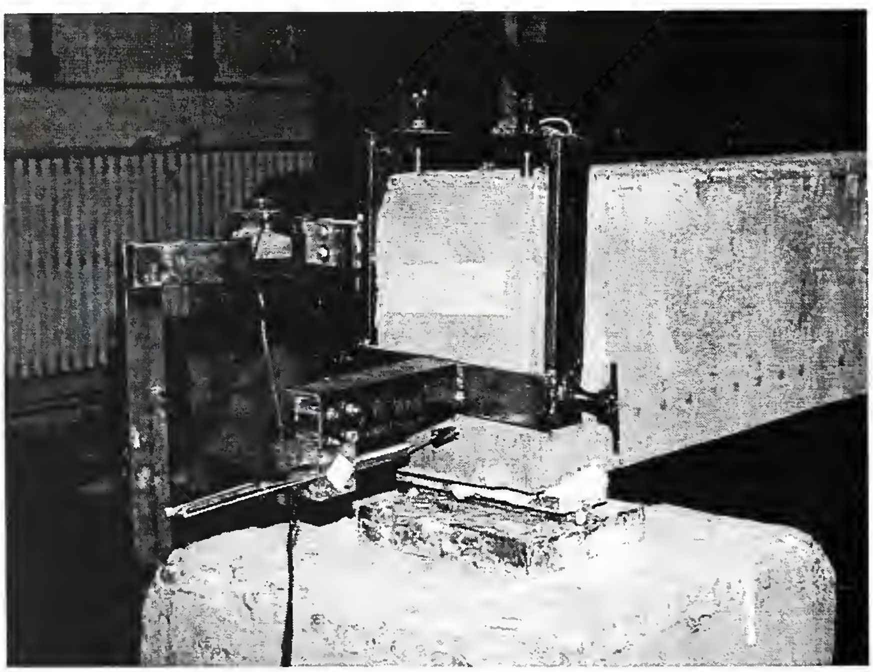


Figure 2.7-5 Photo showing strand slip measuring device for single-strand beams.

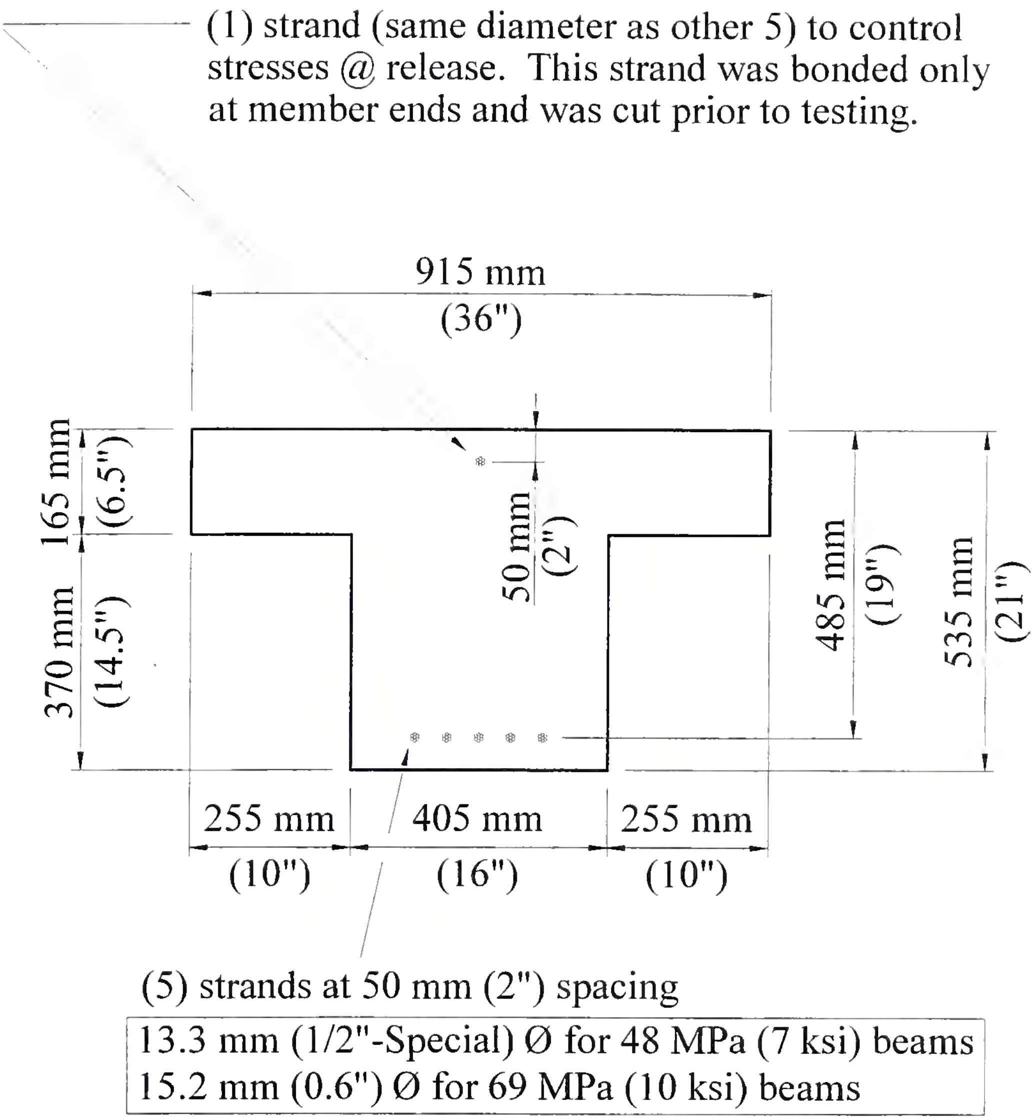


Figure 2.8-1 Cross-section of multi-stranded T-beams.

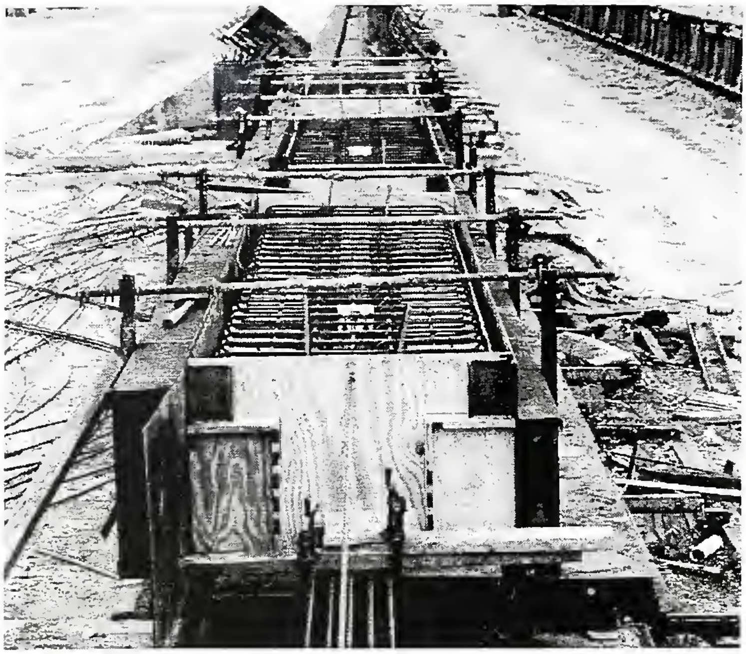


Figure 2.8-2 Photo showing formwork for the 48 MPa (7 ksi) T-beams cast end-to-end.

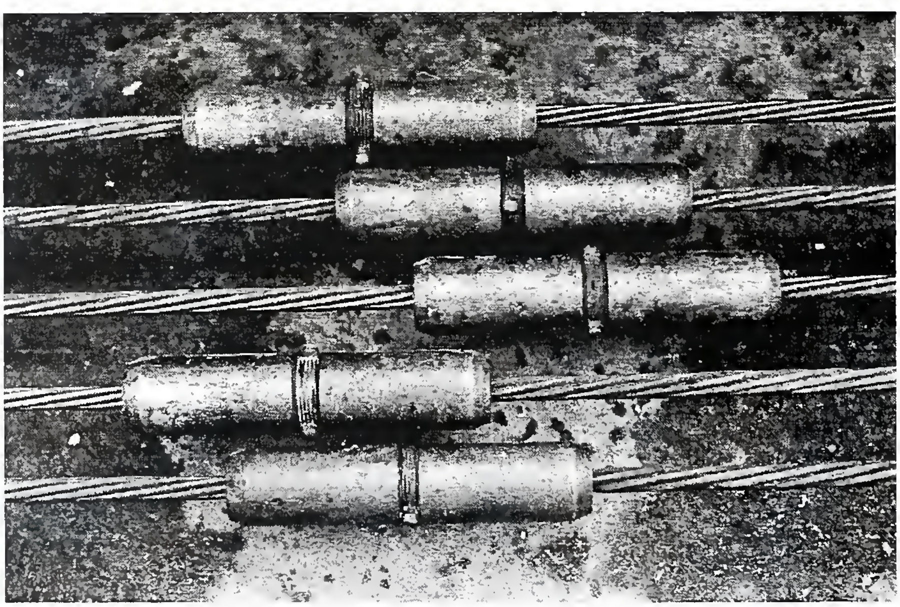
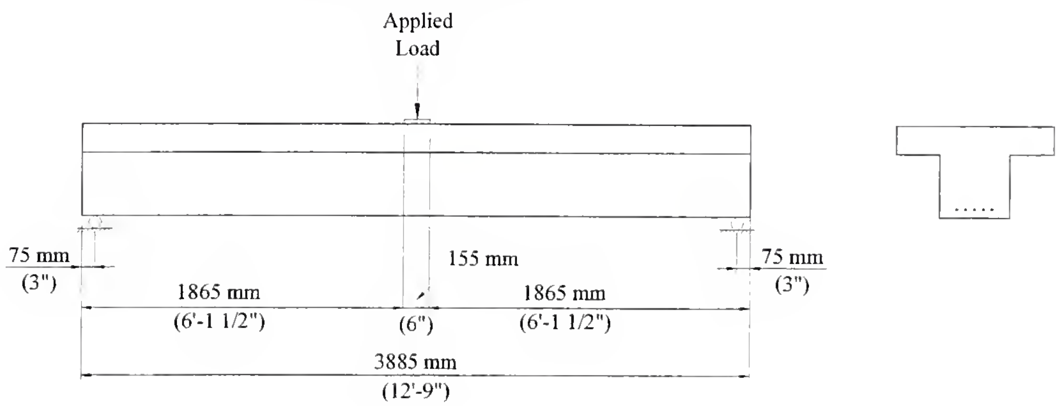


Figure 2.8-3 Photo showing splicing of Insteel and Florida Wire & Cable strand.

48 MPa (7 ksi) T-Beam Loading Arrangement



69 MPa (10 ksi) T-Beam Loading Arrangement

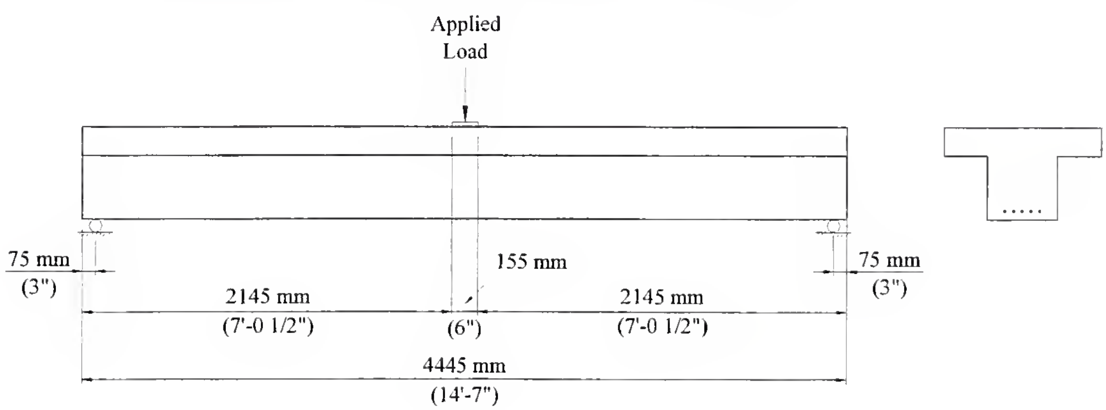


Figure 2.8-4 Loading arrangement for multi-stranded T-beams.

Required Shear Reinforcement

Area of T-beam = 466 in^2

$$\text{wt./ft} = \frac{466\text{in}^2}{\left(\frac{144\text{in}^2}{\text{ft}^2}\right)} \times 140\frac{\text{lb}}{\text{ft}^3} = 451\frac{\text{lb}}{\text{ft}} = 0.451\frac{\text{kip}}{\text{ft}}$$

From Equation (2-3) $\rightarrow f_{ps}$ = 265 ksi

c = 1.47", a = β_1c = 0.70 x 1.47" = 0.987"

Mn = 5 strands x (0.167 in^2/strand) x 265 ksi x (19" - .987"/2) = 4095 kip-in = 341.3 kip-ft

Based on the loading configuration in Figure 2.8-4, the failure load "P" which produces a moment of 341.3 kip-ft at the edge of the 6" loading plate is 113.3 kips. Thus, the maximum shear is approximately

$$\text{Vn} = \frac{113.3\text{ kips}}{2} + \left(.451\frac{\text{kip}}{\text{ft}} \times \frac{12.75\text{ft}}{2}\right) = 59.5\text{ kips (say 60 kips)}$$

Based on the simplified shear design approach in ACI 11.4.1,

$$\text{V}_c = \left(2\sqrt{f_c'}\,b_w d\right) \times 0.85 \text{ for "sand lightweight" concrete} = 2\sqrt{7000}(16")(19") \times 0.85 = 43.2\text{ kips}$$

$V_s = V_n - V_c$ = 60 kips – 43.2 kips = 16.8 kips

ACI (11-15) $\rightarrow$ $V_s = \dfrac{A_v f_y' d}{s}$

Using #4 Ties (Grade 60) -- A_v = 0.40 in^2 , f_y = 60 ksi

Therefore stirrup spacing is calculated ... $s = \dfrac{A_v f_y d}{V_s} = \dfrac{(0.40\text{in}^2)(60\text{ksi})(19\text{in})}{16.8\text{kips}} = 27\text{ inches}$

Minimum spacing requirements control here.

1. $s_{max} = \dfrac{A_v f_y}{50 b_w} = \dfrac{(0.40\text{in}^2)(60{,}000\text{psi})}{(50)(16")}$ = 30 inches based on ACI (11-13)
2. s_{max} = 3/4(h) = 0.75 x 21" = 15.75 inches based on ACI 11.5.4.1
3. s_{max} = 24" based on ACI 11.5.3.1

Therefore, the minimum practical stirrup spacing is 15 inches. ***Use #4 Stirrups at 15" o.c.***

Figure 2.8-5 Calculation of required shear reinforcement in 48 MPa (7 ksi) T-beams.

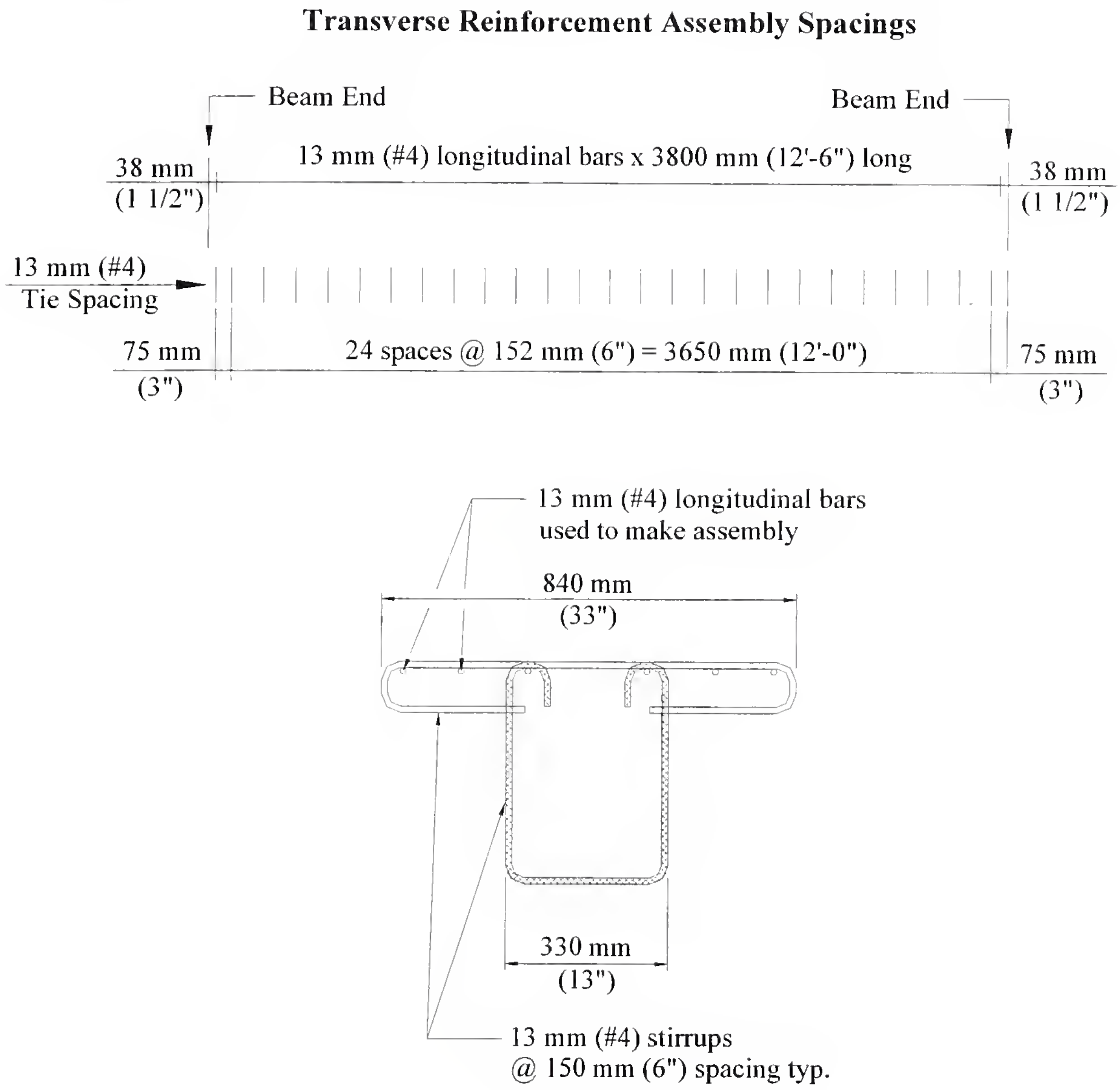


Figure 2.8-6 Transverse reinforcement for 48 MPa (7 ksi) T-beams.

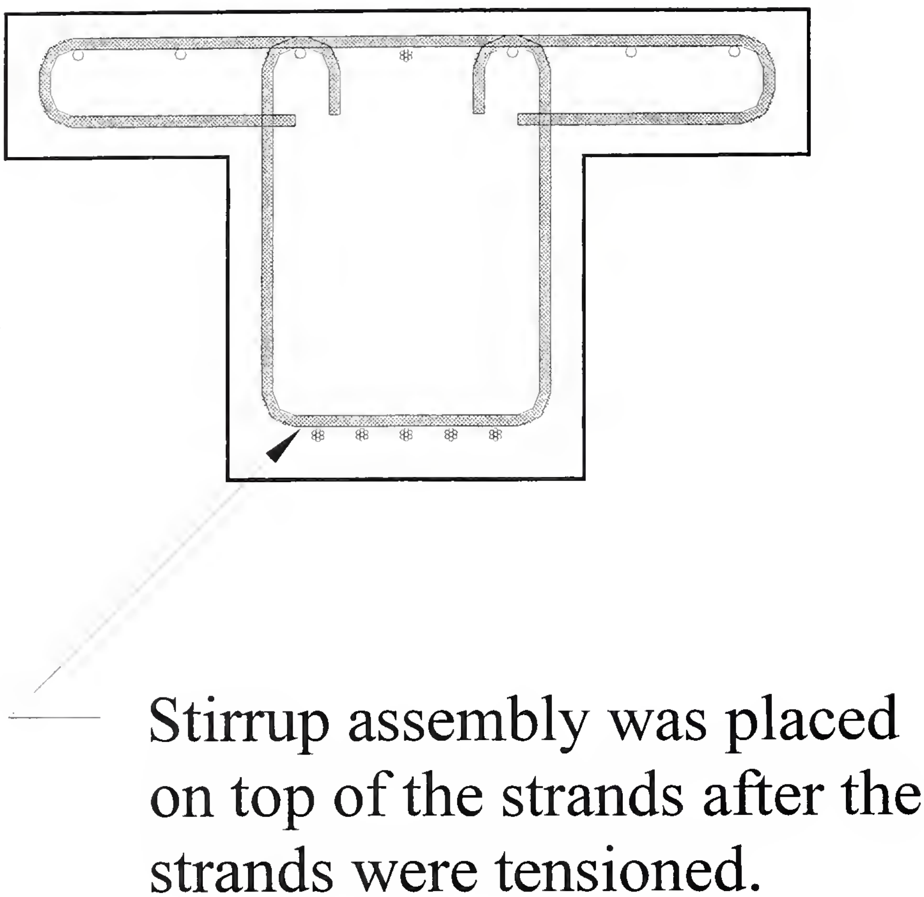


Figure 2.8-7 Positioning of transverse reinforcement for the 48 MPa (7 ksi) T-beams.

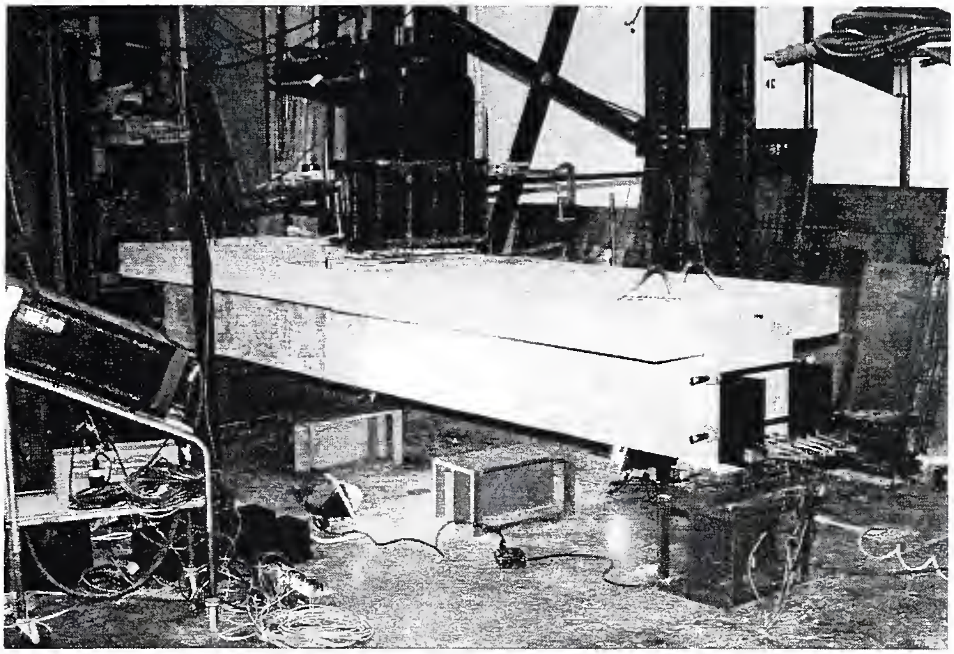


Figure 2.8-8 Test setup for development length evaluation of T-beams.

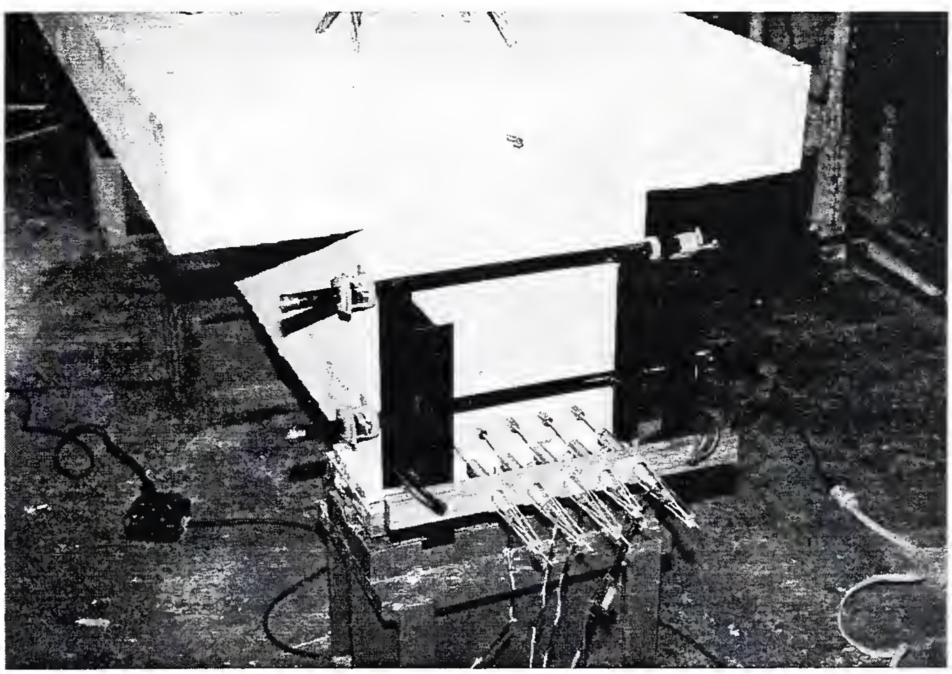
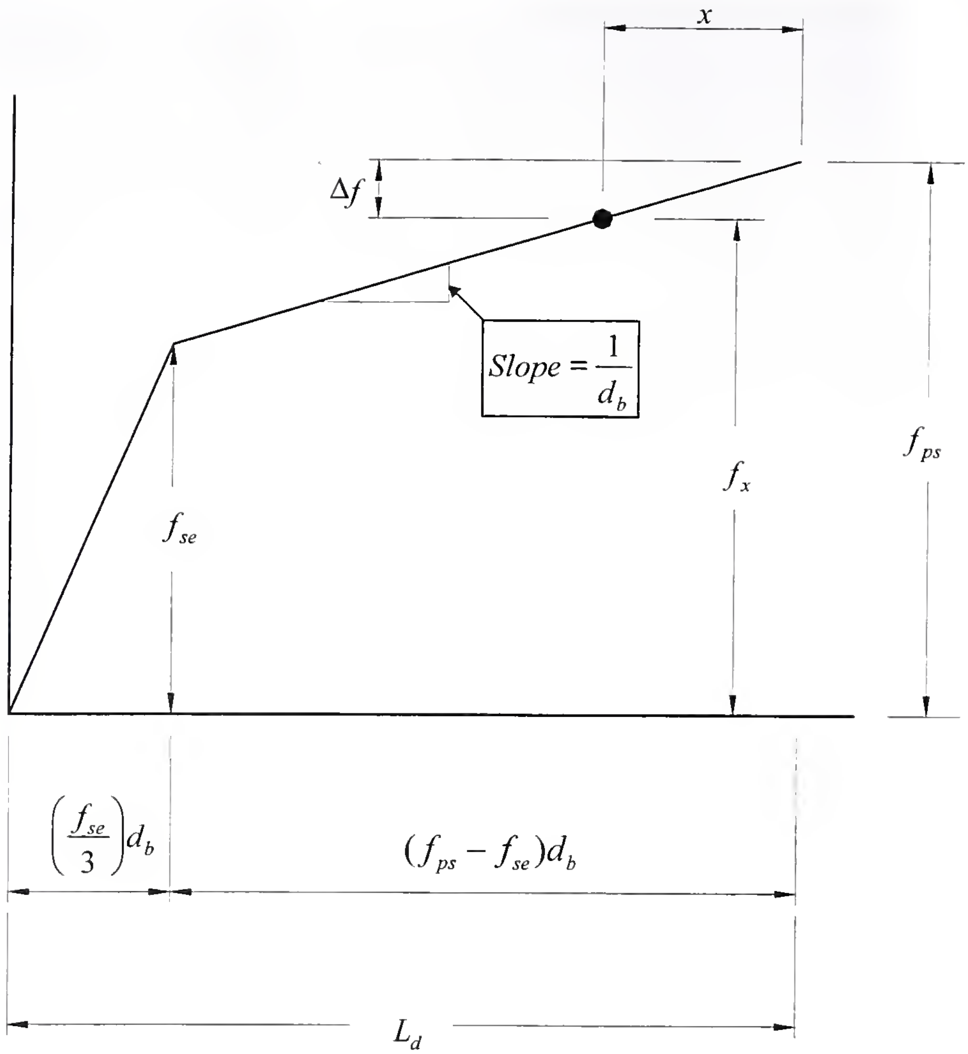


Figure 2.8-9 LVDT's were used to measure the strand slip at both ends of the T-beams.

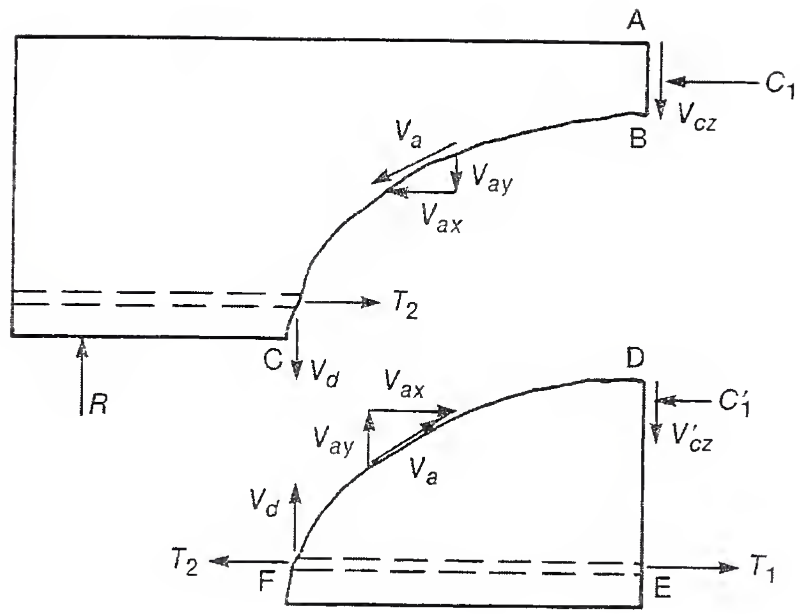


$$Slope = \frac{f_{ps} - f_{se}}{(f_{ps} - f_{se})db} = \frac{1}{d_b}$$

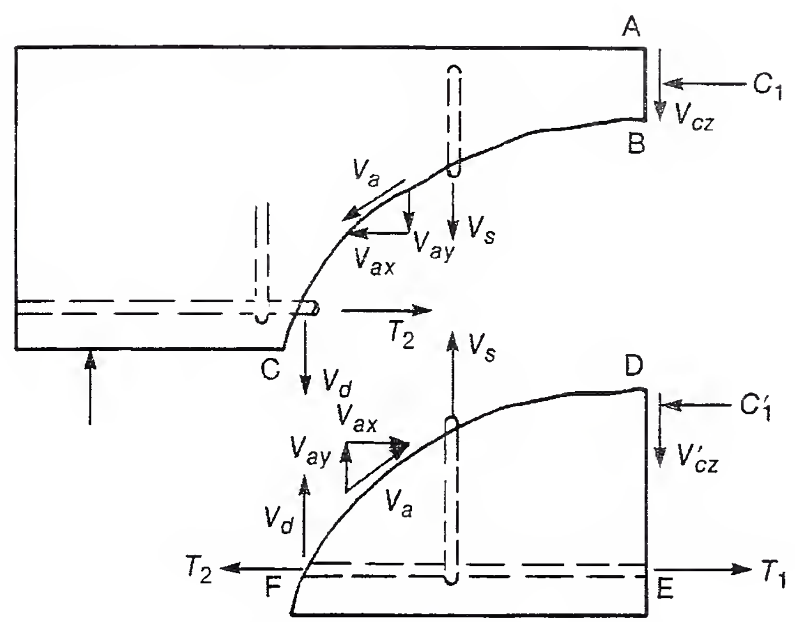
$$\Delta f = Slope \times (x) = \frac{x}{d_b}$$

$$f_x = f_{ps} - \Delta f$$

Figure 2.9-1 Bi-linear variation of steel stress with distance from free end of strand.

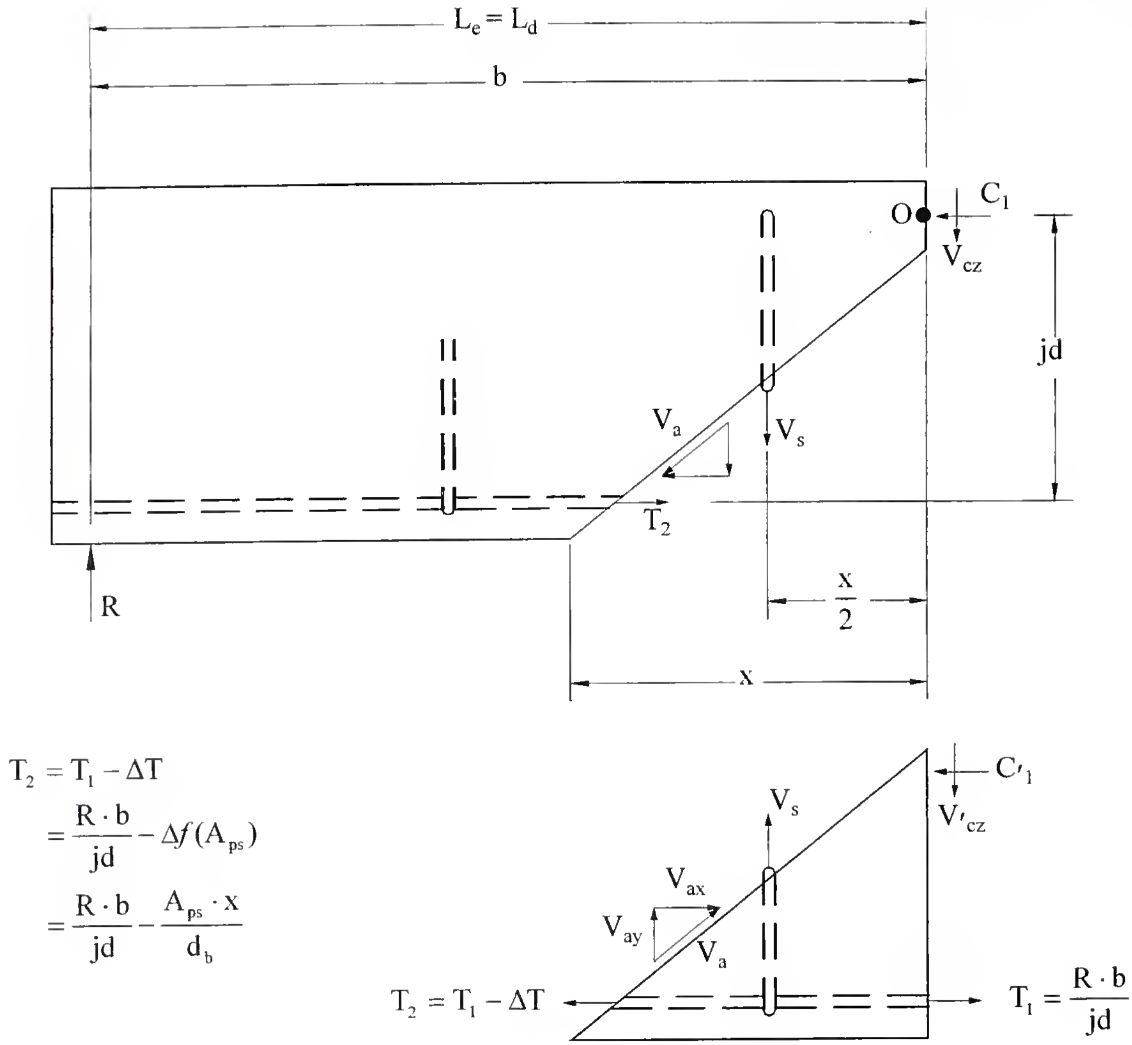


(a) Internal forces in a cracked beam without stirrups.



(b) Internal forces in a cracked beam with stirrups.

Figure 2.9-2 Force distribution in a beam with an inclined crack.



$$
\begin{aligned}
T_2 &= T_1 - \Delta T \\
&= \frac{R \cdot b}{jd} - \Delta f(A_{ps}) \\
&= \frac{R \cdot b}{jd} - \frac{A_{ps} \cdot x}{d_b}
\end{aligned}
$$

<u>Model Assumptions:</u>

- The flexure-shear crack can be represented by a linear crack as shown.
- Dowel action is conservatively ignored.
- The weight of the beam is negligible.
- The line of action of the sum of all aggregate-interlock forces (V_a) passes approximately through point O. Therefore the moment due to this force about point O is small and can be ignored. Note, when ignoring dowel action, the horizontal component of all aggregate interlock forces (V_{ax}) is equal to the change in force in the longitudinal reinforcement (ΔT).

Figure 2.9-3 Model used to determine stirrups needed to reduce tension in longitudinal steel.

From Figure 2.9-3, the force in the stirrups crossing the flexure-shear crack can be determined by summing moments about point O.

$$\circlearrowleft + \quad \Sigma M_O=0 \qquad V_s\left(\frac{x}{2}\right)+\left(\frac{R\cdot b}{jd}-\frac{A_{ps}\cdot x}{d_b}\right)(jd)-R\cdot b=0$$

Solving... $$V_s=\frac{2\cdot A_{ps}\cdot jd}{d_b}$$

Assuming all stirrups crossing the crack are yielding, then the force in the stirrups (V_s) is equal to the total area of the stirrups (A_{vt}) multiplied the yield stress (f_{yv}).

Therefore, $$A_{vt}f_{yv}=\frac{2\cdot A_{ps}\cdot jd}{d_b}$$

$$A_{vt}=\frac{2\cdot A_{ps}\cdot jd}{f_{yv}\cdot d_b}$$

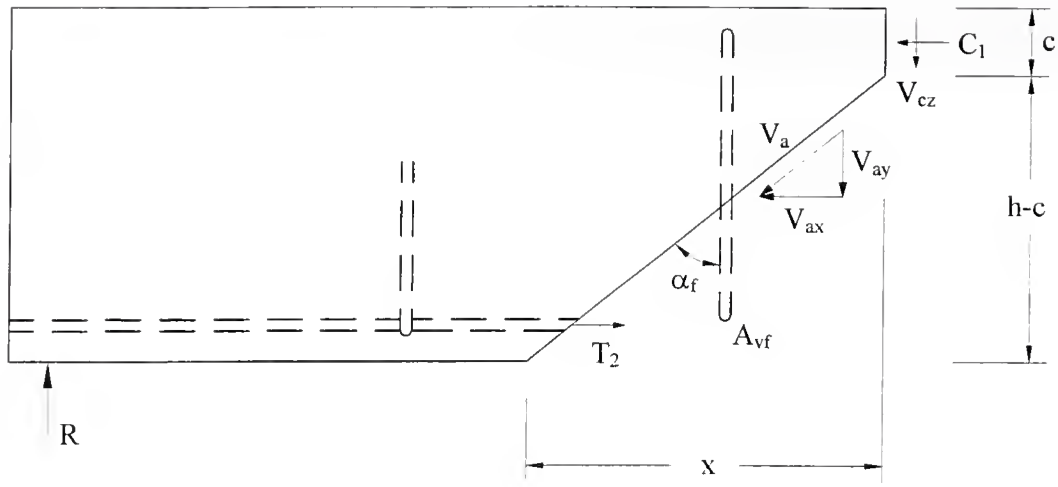
For the 48 MPa T-beams... A_{ps} = (5 strands)(0.167 in^2/strand) = 0.835 in^2 (539 mm^2)
$a = \beta_1 c$ = 0.987 in (from Fig. 2.8-5)
jd = d - a/2 = 19" - .987"/2 = 18.51" (470.1 mm)
f_{yv} = 60 ksi (415 MPa)
d_b = 0.522 in (13.3 mm)

$$A_{vt}=\frac{2\cdot 0.835\,in^2\cdot 18.51\,in}{(60\,ksi)\cdot 0.522\,in}=0.99\,in^2\ (640mm^2)$$

Using #4 (13mm) stirrups (A_v = 0.40 in^2/stirrup), 3 stirrups are required to cross the crack.

Note: Using the model shown in Figure 2.9-3, the area of steel required to cross the crack is independent of the orientation (angle) of the crack. This is because the horizontal force being transferred across the crack (ΔT) increases linearly with the horizontal length of the crack (x), as does the moment arm (from point O) corresponding to the centroid of the resultant stirrup force (V_s).

Figure 2.9-4 Calculation of transverse reinforcement required to reduce the tension force across an inclined crack by the amount ΔT.



LOWER-BOUND ESTIMATE OF REQUIRED STIRRUPS USING SHEAR-FRICTION APPROACH

Horizontal force that must be transferred across crack (V_{ax}) is equal to ΔT.

$$\Delta T = \frac{A_{ps} \cdot x}{d_b} = \frac{A_{ps} \cdot (h-c) \cdot \tan\alpha_f}{d_b} = V_{ax}, \text{ where } \alpha_f = \tan^{-1}\left(\frac{x}{h-c}\right)$$

ACI Equation (11-26) … $V_n = A_{vf} f_y (\mu \sin\alpha_f + \cos\alpha_f)$, $\mu = 1.4\lambda$ for concrete cast monolithically, $\lambda = 0.85$ for "sand-lightweight" concrete

For the beam above, $V_n = V_a = \dfrac{V_{ax}}{\sin\alpha_f}$

$$\text{Therefore, } A_{vf} = \frac{V_{ax}}{f_y \cdot \sin\alpha_f \cdot (\mu\sin\alpha_f + \cos\alpha_f)} = \frac{A_{ps} \cdot (h-c) \cdot \tan\alpha_f}{f_y \cdot d_b \cdot \sin\alpha_f \cdot (\mu\sin\alpha_f + \cos\alpha_f)}$$

For the 48 MPa (7 ksi) T-Beams, h=21", c = 1.47", $A_{ps} = 0.835$ in^2, $f_y = 60$ ksi.

Thus, for a crack at a 45° incline from the horizontal (x = 19.53") the required $A_{vf} = 0.48$ in^2

For a crack at a 65° incline from the horizontal (x = 9.1") the required $A_{vf} = 0.41$ in^2

***Note: There is also a net tension force (T_2) acting across the shear plane that is not accounted for in these calculations. Although this force is carried by the prestressed steel, the flexure-shear crack results in separations between the opposite sides of the crack that are not considered in the shear-friction model. When tension across a crack is present, reinforcement in addition to A_{vf} is required to resist this tension per ACI Section 11.7.7. Therefore, these calculations provide a lower-bound value of the required transverse reinforcement.**

Figure 2.9-5 Calculation of required transverse reinforcement by shear-friction.

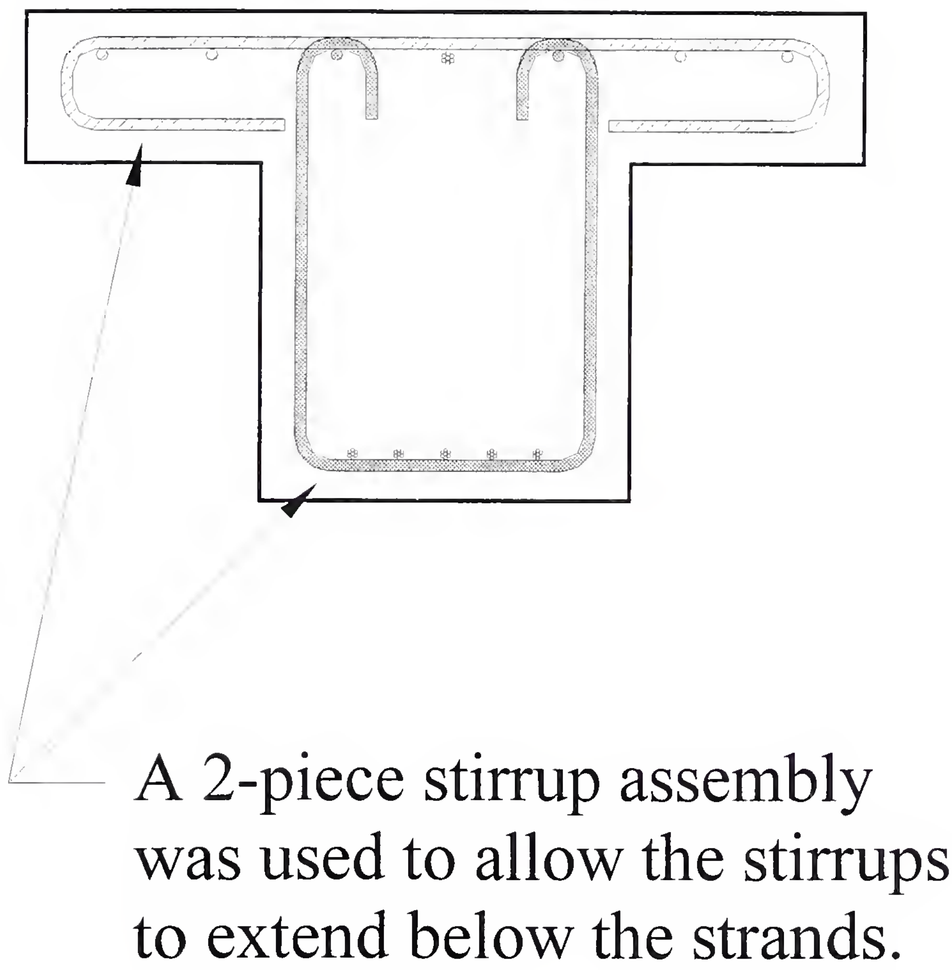


Figure 2.10-1 Positioning of transverse reinforcement for the three additional 48 MPa (7 ksi) T-beams plus the 69 MPa (10 ksi) T-Beam.

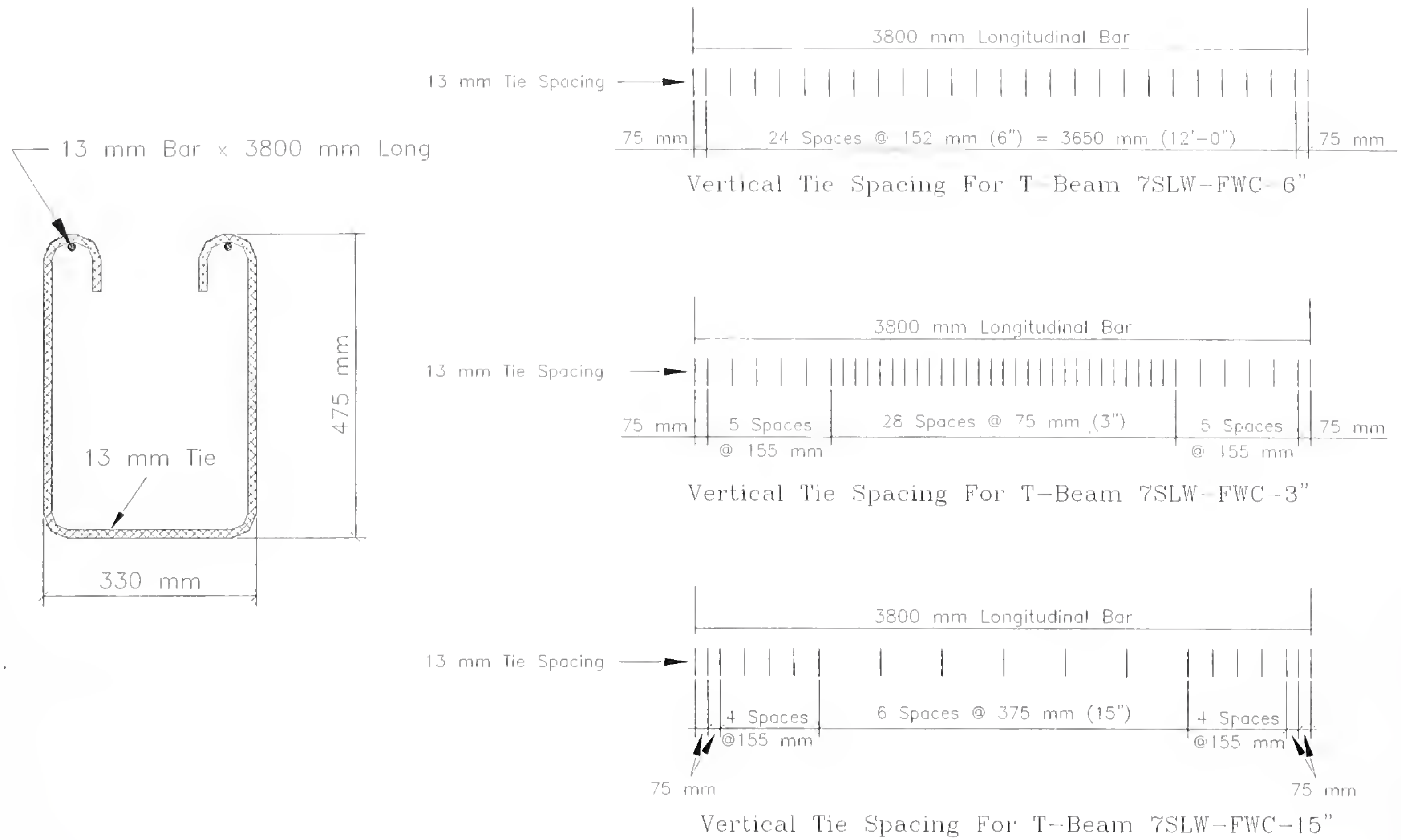


Figure 2.10-2 Vertical stirrup detailing for the three additional 48 MPa (7 ksi) T-Beams

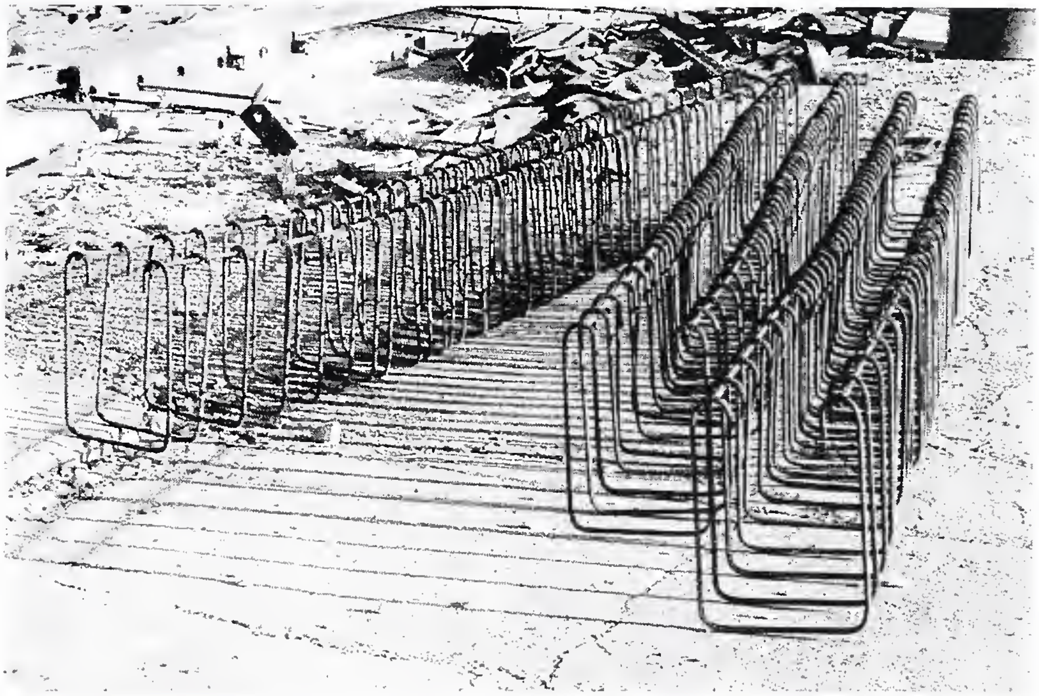


Figure 2.10-3 Photo showing stirrup assemblies that were placed below longitudinal strands.

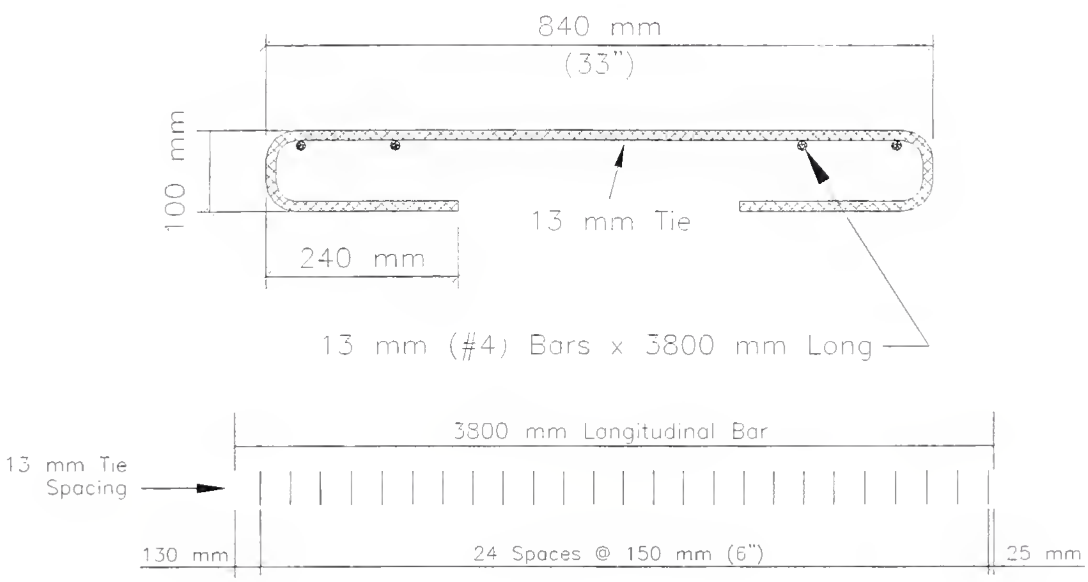


Figure 2.10-4 Flange reinforcement for the three additional 48 MPa (7 ksi) T-beams.

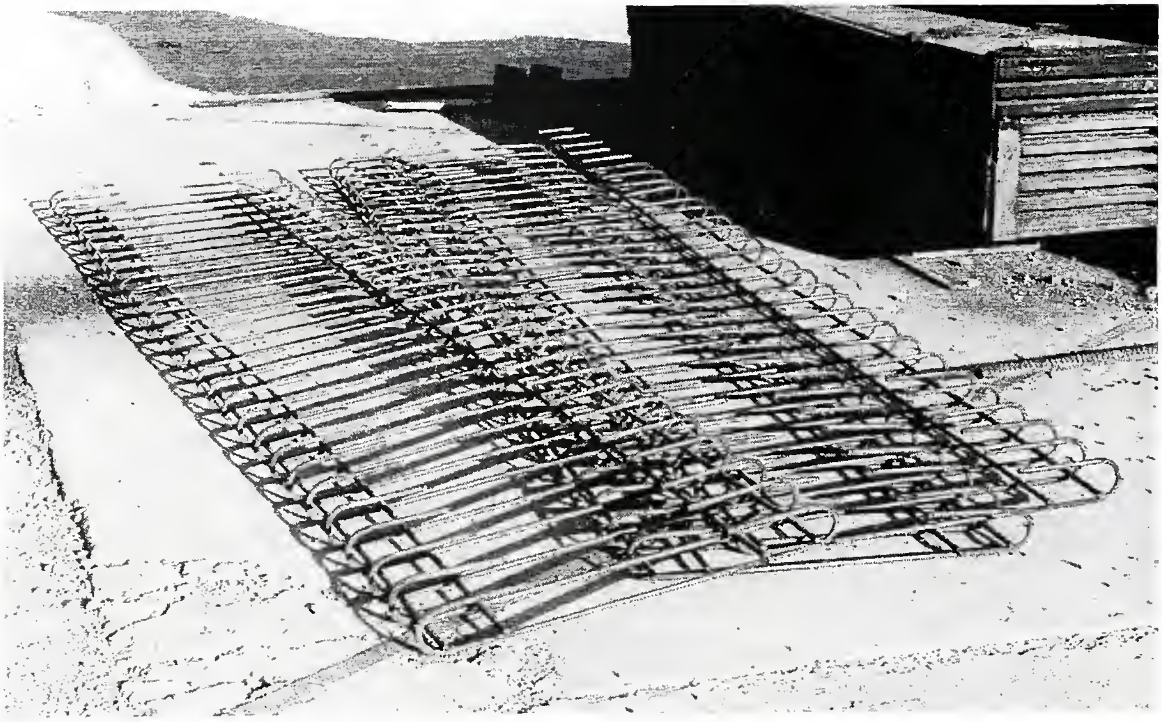


Figure 2.10-5 Photo showing reinforcement used in the flanges of the additional T-beams.

CHAPTER 3 – EXPERIMENTAL RESULTS

3.1 Pullout Tests

A total of four Moustafa pullout specimens were fabricated and tested as part of this experimental program (refer to Table 2.1). Pullout Specimen #1 was based on the standard 7-bag concrete mix shown in Appendix A, and was used to determine the bond quality of the strand at the beginning of the study (i.e. if the strand met the minimum pullout load of 160 kN recommended by Logan [3]). The specimen contained eighteen 13.2 mm (1/2"-Special) strands -- nine of the strands were produced by Florida Wire & Cable, while the other nine were produced by Insteel.

Pullout Specimen #1 was cast on 9/16/97 and tested on 9/18/97 (at 2 days) and on 9/20/97 (at 4 days). The strands were pulled out of the block using a hydraulic ram furnished by CSR Hydro-Conduit (see Figure 3.1-1). The load was recorded using a load cell that was placed between two steel plates. The bottom plate had two steel angles welded to the bottom side to allow the same contact area with the concrete block that was specified by Logan (refer to Figure E3 of Appendix A). Figure 3.1-2 shows the load cell configuration used to determine the pullout values. The maximum load occurring during a given pullout test was stored automatically by the data acquisition system. Also, the load at "first slip" was obtained by placing a small piece of tape near the point where the strand entered the concrete. As soon as a motion was detected, the person monitoring the slip simply pressed a "mouse" button and the computer also recorded this value.

Figure 3.1-3 shows the pullout loads, "first slip" values, and the positioning of the strands in Pullout Specimen #1. As indicated, the average pullout capacity for each strand type tested exceeded the minimum value of 167 kN (37.6 kips), indicating that the bond quality of both strands was acceptable. Figure 3.1-3 shows that the pullout and first-slip values were slightly higher at four days than at two days, as expected, due to the higher concrete strength and presumed higher modulus of elasticity. This figure also shows that although both strands had similar ultimate pullout capacities, the Florida Wire & Cable strand consistently began to slip at a lower load.

Pullout Specimen #2 utilized strands from the same rolls and producers as Pullout Specimen #1, but was cast with a 48 MPa (7 ksi) normal-weight (NW) concrete instead of the standard "PCI Mix" specified in Appendix A. The properties of the concrete mix used for this specimen, as well as all of the other specimens in this study, are located in Appendix B. This specimen was cast on 9/18/97 and pullout tests were done on 9/20/97 (at 2 days). Figure 3.1-4 shows the pullout loads, "first slip" values, and the positioning of the strands in Pullout Specimen #2.

From Figure 3.1-4 it can be seen that although the mix had a target strength of 48 MPa at 28 days, the strength at two days had already exceeded this and was 58.6 MPa (8500 psi). Interestingly, though, the pullout values for this specimen at two days were consistently lower than the two-day pullout values from Pullout Specimen #1, which had a two-day compressive strength of only 33.1 MPa (4800 psi). In fact, only one of the eighteen strands in this specimen met the minimum value of 167 kN. It is evident that concrete properties, independent of strength, can significantly affect the results of the Moustafa test.

Pullout Specimen #3 utilized strands from the same rolls and producers as Pullout Specimens #1 and #2, but was cast with a 48 MPa (7 ksi) semi-lightweight (SLW) concrete instead of the standard “PCI Mix” specified in Appendix A. This specimen was cast on 10/3/97 and pullout tests were conducted on 10/7/98 (at 4 days) and on 2/28/98 (at 148 days). Figure 3.1-5 shows the pullout loads, “first slip” values, and the positioning of the strands in Pullout Specimen #3. This figure shows that, again, a different concrete mix resulted in pullout values at four days that were below the minimum recommended value of 167 kN (37.6 kips). This was true even thought the compressive strength for Pullout Specimen #3 was higher than that for Pullout Specimen #1 at two days (48.3 MPa verses 33.1 Mpa). These results are not as surprising as the ones for Pullout Specimen #2, because Pullout Specimen #3 contained light-weight aggregate and therefore would likely have a lower modulus of elasticity. However, the results support the notion that concrete properties other than compressive strength can significantly influence the results of the Moustafa test.

Pullout Specimen #4 utilized a 69 MPa (10,000 psi) concrete mix and contained both 13.2 mm (1/2”-Special) Insteel strands and 15.2 mm (0.6”) Insteel strands. This specimen was cast on 10/28/97 but was not tested until 6/18/98 (233 days later) due to a breakdown in the load cell and data acquisition system. The compressive strength of the concrete in this specimen tested at over 96 MPa (14,000 psi) at 91 days of age. Due to the higher capacities with the larger strand, a different pulling device had to be used. Figure 3.1-6 shows the test configuration used to pull the strands in Pullout Specimen #4. A center-hole post-tensioning ram was used along with the steel plates and load cell arrangement used during tests on Pullout Specimens #1, #2, and #3.

Figure 3.1-7 shows the pullout loads, “first slip” values, and the positioning of the strands in Pullout Specimen #4. Pullout values for all strands were very high, due probably to both the high-strength mix and age of the concrete. Three of the strands (IST-28, IST-32, and 0.6-#8) ruptured during the test. This failure mode was typical for some of the pullout specimens tested by Logan. Except for the three strands mentioned above, and for strand IST-2 in Pullout Specimen #1 which also failed by strand rupture, the pullout specimens in this study failed by slip, and failures were almost always associated with load popping sounds.

3.2 Surface Condition Assessment

From the outset of this study it was noted by several observers that the two 13.3 mm (1/2"-Special) strands used had a markedly different appearance. The Insteel strand had a bluish hue about it that might best be described as a "gun-steel blue" color. The strand form Florida Wire & Cable, on the other hand, had a yellow/brassy tint.

Prior to casting the pullout specimens, the strand samples were wiped with a paper towel to remove residue and help assist in the visual assessment. Figures 3.1-1 and 3.2-1 show the towels used to wipe the strands for Pullout Specimens #1 and #2, respectively. Towels used to wipe the strands in the other pullout specimens appeared similar. These figures show there is a distinct difference between the residue on the strands. The towels from the Florida Wire & Cable strand have considerably more noticeable residue that those corresponding to the Insteel strands. In performing the towel wipes, it was noted that it was easier to wipe the Florida Wire & Cable strands than the Insteel strands, as there was much more of a tendency to bind or tear the towels on the Insteel strands when applying equal pressure. In general, the residue corresponding to the Insteel strands was brown or rust colored while the residue corresponding to the Florida Wire & Cable strands was black. The chemical composition of this residue was not determined.

It should be noted that all "towel wipes" in this study were performed by the same person.

Also, the strand rolls in this study were placed under cover when they were received from the producer in attempt to minimize weathering. All 13.3 mm (1/2"-Special) strand was purchased for this project through CSR Hydro-Conduit. The 15.2 mm (0.6") strands were donated by Insteel.

3.3 Measurements of Transfer Length

Values of surface strains were measured on specimens similar to those in Figure 2.4-2. The transfer length can then be inferred directly from the recorded values of strain. For a concentrically prestressed member supported along the entire length, such as the one in Figure 2.4-2, the member will be under simple axial loading when the prestress force is transferred to the concrete. Thus, the compressive stresses (and hence strains) in the member will vary from zero at the free end to a constant value corresponding to the "P/A" stress. The length it takes for the strand to bond to the concrete and "transfer" the full tension in the cable to the concrete is called the transfer length.

Figure 3.3-1 shows the results of surface strain measurements taken for the 48 MPa (7000 psi) SLW transfer-length specimen using 13.3 mm (1/2"-Special) Insteel strand. Measurements were taken immediately after transfer of prestress, and at 3, 14, 36, and 66 days thereafter. The vertical dashed lines are drawn at the approximate breaks between the "sloping" portion of the curves where strains are increasing and the "flat" portion corresponding to the region under constant stress of P/A. The distance from the end of the transfer-length specimen to the dashed line is the transfer length.

Figure 3.3-1 shows that the measured surface strains were not symmetric with respect to the center of the beam. This is because the left-hand-side (as plotted) of the beam developed a longitudinal crack upon cutting the prestressed cables and transferring the prestress force. Thus, the longitudinal crack allowed additional slip to occur between the strand and concrete and the transfer length increased. This is a problem when using small transfer-length specimens and detensioning by flame cutting the strands. The advantage of the small specimens, however, is that they provide higher compressive strains than typical beams so the measured are more reliable.

Observation of the right-hand side of Figure 3.3-1 reveals that the transfer length would be approximately 500 mm (20"). This corresponds to 38 strand diameters for a 13.3 mm strand, less than the assumed 50 strand diameters used in the ACI Code for checking shear provisions (ACI Section 11.4.4). Thus, the combination of 13.3 mm Insteel strand and 48 MPa SLW concrete would meet code requirements for transfer length.

Figure 3.3-1 also clearly shows the increase in compressive strains due to creep. Shrinkage also causes a similar shortening of the concrete specimens, but "dummy" readings were taken on a small non-prestressed concrete beam cast at the same time and with the same concrete as the transfer-length specimens. These readings were then used to remove temperature effects from the readings and they also served to remove the shrinkage components as well. By inspection of Figure 3.3-1, it can be seen that although the strains continue to increase with time due to creep, the actual transfer length remains essentially unchanged.

Figure 3.3-2 shows the results of surface strain measurements taken for the 48 MPa (7000 psi) SLW transfer-length specimen using 13.3 mm (1/2"-Special) Florida Wire & Cable strand. Measurements were taken immediately after transfer of prestress, and at 3, 14, 36, and 66 days thereafter. The vertical dashed lines are drawn at the approximate transfer length from each end.

Figure 3.3-2 shows that the transfer length at each end of the beam is approximately 550 to 600 mm (21 ½ to 23 ½ inches), clearly less than the 665 mm (26") distance corresponding to 50 strand diameters for the 13.3 mm (1/2"-Special) strand. Thus, the combination of 13.3 mm Florida Wire & Cable strand and 48 MPa SLW concrete would meet code requirements for transfer length.

A transfer-length specimen, similar to the two discussed above, was also fabricated with the combination of 69 MPa (10,000 psi) concrete and 15.2 mm (0.6") strand. However, upon transfer of prestress the specimen developed longitudinal cracks that extended for nearly the entire length of the beam, as the tensile capacity of the concrete was less than the demand required to restrain the expanding 15.2 mm (0.6") strand plus coupled with impact due to flame cutting. Thus, the specimen was rendered useless for transfer-length measurements.

Section 2.4 of this report mentioned that transfer-length measurements were taken for some of the single-strand development-length specimens. Most of these readings were unreliable, due to weather considerations and the small magnitude being measured. Nonetheless, some of these measurements were useful for qualitative purposes. In particular, the highest levels of compressive strains in these specimens would correspond to those using the largest strand, namely 15.2 mm (0.6") strand.

Figure 3.3-3 shows the surface strains recorded at one end of beam 10SLW-0.6-1S. The "zero" point on this graph has been arbitrarily adjusted as shown, since the effects of rapidly changing temperatures in late October made absolute strain determinations futile. However, Figure 3.3-3 shows that the prestress force has been fully transferred within 600 inches from the end of the beam. In this figure, the vertical dashed line indicating the place corresponding to the transfer length has been conservatively positioned. The Code assumed transfer length corresponding to 50 strand diameters is 760 mm. Thus, it appears that the actual transfer length for the 15.2 mm strand in 69 MPa concrete meets code requirements.

3.4 Results From Single-Strand Development-length Tests

A total of eighteen (18) single-strand development-length tests were conducted in this study (9 beams tested at both ends). As discussed in Section 2.7, the single-strand beam specimens provided a cost-effective means of conducting many load tests. Appendix C contains the load-deflection plots recorded for each of the eighteen load tests to failure. Photographs of the specimens during testing, and documentation of the crack patterns occurring in the beams, are contained in Appendix D. The failure loads and deflections corresponding to the maximum sustained load are shown in Figures 3.4-1, 3.4-2, and 3.4-3.

Figure 3.4-1 gives the test data, in metric units, for the 48 MPa (7000 psi) single-strand beams with 13.3 mm (1/2"-Special) strands from both Insteel and Florida Wire & Cable. Figure 3.4-2 contains the same information as Figure 3.4-1, except the data are presented in U.S. customary units. These two figures also show the maximum moment in the beams, occurring at a distance L_d from the end of the beam and determined from the measured values of applied load. In every case, the maximum moment withstood by the specimen exceeded the AASHTO nominal moment capacity (Mn), indicating the beams' strands were adequately developed at a distance from the end of the beam equal to the code-prescribed development length L_d. This is consistent with the results from measurements of strand end-slip during testing, which revealed that slip did not occur in all but one specimen, namely 7SLW-IST-1S. In this specimen, a strand slip of 1.3

mm (0.051 in) was recorded on the data scan prior to failure. However, this minimal slip occurred after the nominal moment capacity had been exceeded by over 10 percent.

Figures 3.4-1 and 3.4-2 also indicate that all failures occurred after considerable deflections had occurred. Each of the 48 MPa beams deflected more than 40 mm (1 1/2") in a 4650 mm (15'-3") span prior reaching its ultimate capacity. Eight of the twelve beams failed when the strands ruptured. The other four specimens failed in shear. While the shear failures occurred when the shear stress on the section (V/bd) was less than 92 psi, or $1.1 \times \sqrt{f_c'}$, they occurred well after yielding of the prestressing steel had occurred and considerable ductility exhibited. The investigators speculate that when the prestressing steel yielded, the effects of dowel action diminished and the lightly-reinforced beams (without stirrups) became susceptible to shear.

Figure 3.4-3 lists the test data, in both metric and U.S. customary units, for the 69 MPa (10,000 psi) single-strand beams with 15.2 mm (0.6") Insteel strands. As in the case of the 48 MPa single-strand beams, all of the specimens were able to reach and exceed the AASHTO nominal moment capacity prior to failure. In addition, measurable strand slip did not occur for any of the 69 MPa beams. Figure 3.4-3 shows that all of the 69 MPa beams exhibited excellent ductility, deflecting more than 57 mm (2 1/4") in a 4955 mm (16'-3") span prior to reaching their ultimate load-carrying capacity. Thus, the AASHTO and ACI development lengths were adequate to develop the 15.2 mm (0.6") strands in the high-strength (69 MPa) SLW concrete single-strand beams.

In summary, test results from the eighteen single-strand development length specimens indicated that the code required development lengths were ample do develop the capacity of a single pretressed strands in a member cast with semi-lightweight (SLW) concrete. Therefore, as discussed in Section 2.8, the multiple-stranded T-Beam tests were also designed based on the current ACI and AASHTO development lengths. The results of the T-Beam specimens are discussed in Section 3.5.

3.5 Results From Development-length Tests On Multi-Stranded T-Beams

A total of seven multi-stranded T-Beams were tested in this study to determine the adequacy of applying the current AASHTO and ACI development-length equations to members with semi-lightweight (SLW) concrete. Results from eighteen development-length tests on single-strand beams indicated that the current code required development lengths were conservative for SLW members with a single prestressed strand. Therefore, the sufficiency of these requirements when applied to SLW members with multiple strands was also tested.

Six of the T-Beams contained 13.3 mm (1/2"-Special) strands, as this is the most commonly used strand for current bridge projects in Indiana. The purpose of the seventh specimen was to investigate the use of a higher-strength (69 MPa) SLW concrete. In order to take advantage of higher concrete compressive strengths in design, it is also necessary to increase the tension capacity (i.e. prestress force) in the member. Thus, in anticipation of future design needs, the 69 MPa (10 ksi) T-Beam utilized 15.2 mm (0.6") strands.

Figure 3.5-1 summarizes the test data, including the failure loads, deflections corresponding to the maximum sustained load, and maximum moments for the 48 MPa (7000 psi) SLW T-Beams. Figure 3.5-2 contains similar data for the 69 MPa (10,000 psi) T-Beam. Appendix C contains the load-deflection plots recorded for each of the seven T-Beam load tests to failure. Photographs of the T-Beam specimens during testing, and the corresponding documentation the crack patterns, are contained in Appendix E.

As discussed in Section 2.10, the first three T-Beam specimens cast, namely T-Beams IST1, IST2 and FWC were identical in detail. Figure 3.5-1 shows that both T-Beams containing Insteel strand, IST1 and IST2, exceeded the AASHTO nominal moment capacity for the section and failed by strand rupture. Thus, the AASHTO and ACI development lengths were sufficient for these beams. These failures were very ductile, as the mid-span deflections exceeded 36 mm (1 3/8 inches) prior to the ultimate capacity being attained. T-Beam FWC had essentially the same ultimate capacity as T-Beam IST1 of 536 kN (120.5 kips). Interestingly, though, this load corresponded to a much lower deflection for the FWC beam (20.8 mm vs. 51.2 mm) and the FWC beam failed by bond, as the strands slipped with respect to the surrounding concrete and pulled in from the west end of the beam (see Figure 3.5-3).

Figure 3.5-4 shows the values of strand slip (for all five strands) with respect to the west-end of T-Beam FWC that were recorded just prior to collapse. In this figure the strand-slip values are listed in the order of strand positioning in the beam, with “C” denoting the middle strand. Note that strand “C” began to experience considerable slip at the maximum load of 120.5 kips (536 kN). Slip then progressed (almost systematically) to the outer strands until total collapse occurred.

After reviewing videotape of the failure of T-Beam FWC the investigators noted that a flexure-shear crack developed just prior to collapse (see Figure 3.5-5). This observation led to the hypothesis that the flexure-shear crack caused an increase in tension nearer the beam end and effectively shifted the “critical section” from the section at the point load to the place where the flexure-shear crack intersected the strand. This was discussed, in detail, in Section 2.9.

It is important to note that the cracking that occurred in T-Beam FWC prior to failure was not dissimilar to the crack patterns that developed in T-Beams IST1 and IST2 (refer to Appendix E for cracking documentation for all T-Beams in this study), yet significantly different modes of failure occurred. The researchers speculated (based partly on pullout specimen behavior and towel-wipe tests) that the bond quality of the Insteel strand may cause it to develop over a shorter distance than the equivalent-sized Florida Wire & Cable strand. If this were true, than a flexure-shear crack which shifts the tension demand closer to the support may not be critical in the case where Insteel strand was used. In general, this tension shift would only lead to sudden collapse upon cracking if the actual distance required to develop a strand lies between the point of maximum moment and the point where the diagonal crack intersects the strand.

In order to test the hypothesis outlined above, three additional 48 MPa (7000 psi) T-Beam specimens utilizing 13.3 mm (1/2"-Special) FWC strand were fabricated and tested, each having the same dimensions and test configuration at the original three T-Beams. These specimens differed from one another by the amount of transverse reinforcement near mid-span (refer to Section 2.10). In addition, the detailing of the vertical stirrups was changed so that the stirrups for the additional three beams would encase the longitudinal strands, unlike the original three T-Beams. The reasoning for this becomes clearer when examining Figures 3.5-5 and 3.5-6.

Figure 3.5-5 shows a horizontal crack that extends towards the end of the beam at the approximate level of the strand. Review of videotape showed that this crack occurred subsequent to the flexure-shear crack opening. Figure 3.5-6 is a view of the opposite (North) side of T-Beam FWC. This figure shows that the strand has been exposed for several feet as a result of the failure mechanism in the beam. With closer inspection of Figure 3.5-6, one can see the vertical stirrups that are "resting" on top of the exposed strands. If a truss model is envisioned for the internal load paths in the beam, then the effect of the stirrup detailing used in the original three T-Beams is to construct a truss and then remove all of the pins connecting the vertical tension members to the bottom tension chord. The result is that the beam is forced to behave as a tied arch if horizontal cracking occurs and the full tension force is demanded near the ends of the beam. Therefore, when constructing the additional specimens, two-piece stirrup assemblies were used which allowed the vertical stirrups to be positioned underneath the longitudinal strands (refer to Figures 2.10-1 and 2.10-3).

Figure 3.5-1 shows the results from the addition three 48 MPa (7000 psi) T-Beam tests. T-Beam FWC-6" was tested first. This beam had a constant 152 mm (6") stirrup spacing (refer to Figure 2.10-2 and was identical to T-Beam FWC, except for the detailing of the vertical stirrups that was discussed in the previous paragraph. As Figure 3.5-1 indicates, T-Beam FWC-6" failed by bond / shear at a load of 489.2 kN (110.0 kips), corresponding to 97.6 percent of the AASHTO nominal moment capacity for the section (see Appendix E for photos of this beam test). Strand slip data sowed that all strands had small values of slip, with the largest slip recorded prior to failure being 0.84 mm (0.033 in).

At the time of failure, the load was held constant and crack patterns were being recorded. Therefore, it is likely that additional slip of the strands occurred during the time period when the load was held constant and end-slip readings were not continuously recorded. While it cannot be proven, it is plausible that additional slip of the strands resulted in a reduced prestress force and therefore a loss in shear capacity. Figures 3.5-7 and 3.5-8 show the failure cracks and end slip for

T-Beam FWC-6", respectively. Figure 3.5-7 also shows the stirrups extending below the longitudinal strands in the member.

T-Beam FWC-3" was the next beam tested. This beam had 13 mm (#4) stirrups at 75 mm (3") on center in the middle portion of the beam (refer to Figure 2.10-2). Figure 3.5-1 shows that this beam failed by strand rupture at a load of 577.7 kN (129.9 kips). This load corresponded to a maximum moment in the beam that was 14.7% larger than the AASHTO nominal moment capacity for the section. Review of strand-slip data for this beam showed that slip was essentially zero at the time of failure. Nine of the ten ends measured had a recorded slip at failure that was less than 0.03 mm (.001 in). The other strand had a recorded slip of 0.12 mm (0.046 in).

T-Beam FWC-3" had the same strand and concrete batch used in T-Beam FWC-6", which experienced bond failure at a load of only 489.2 kN (110.0 kips). In other words, with stirrups spaced at 75 mm (3") on center, T-Beam FWC-3" was able to withstand an applied load that was 18.1% larger than the failure load for T-Beam FWC-6". Figure 3.5-9 shows the failure crack and corresponding strand rupture for T-Beam FWC-3".

T-Beam FWC-15" was the last beam tested in the 48 MPa (7000 psi) series. This beam had a stirrup spacing in the central region of the beam of 375 mm (15"), which corresponded to the ACI Code minimum amount required for shear. As expected, this beam experienced bond / shear failure at only 444.9 kN (100.0 kips), the lowest load for all the 48 MPa (7000 psi) T-Beam specimens tested (see Figure 3.5-1). Figure 3.5-10 shows the shear failure that occurred after strand slip initiated in the member.

Figure 3.5-2 shows that the 69 MPa T-Beam with 15.2 mm (0.6") Insteel strands failed by strand rupture at 622.8 kN (140.0 kips). This value was 9.8% greater than the AASHTO nominal moment capacity for the section of 601.2 kN-m (443.6 kip-ft). Since there was only one T-Beam tested that used the higher concrete strength and larger strand diameter, the investigators decided to use 13 mm (#4) stirrups at 75 mm (3") spacing throughout the central portion of the beam. Also, the transverse reinforcement in T-Beam 10SLW-0.6 utilized the detail in which the stirrups enclosed the strands.

In summary, both of the 48 MPa (7000 psi) T-Beams containing Insteel strand (IST1 and IST2) experienced flexural failures (by strand rupture). Each had 13 mm (#4) stirrups at 152 mm (6") throughout the entire length of the beam and used the undesirable transverse reinforcement detail (where the stirrups did not surround the longitudinal strands). Three of the four T-Beams utilizing Florida Wire & Cable strand (FWC, FWC-6", and FWC-15") experienced bond failure and (believed subsequent) shear failure when loaded at a distance from the end of the beam equal to the AASHTO and ACI development lengths. These failures occurred suddenly, and without much warning, at significantly smaller deflections (refer to Figure 3.5-1 and the load-deflection plots in Appendix C). Flexural failure (by strand rupture) was achieved in a T-Beam using Florida Wire & Cable strands when 13 mm (#4) stirrups were provided at 75 mm (3") centers in the middle portion of the beam. This spacing provided a stirrup area that was five times greater than the amount required by ACI shear provisions (see Figure 2.8-5). T-Beam 10SLW-0.6, which contained 15.2 mm (0.6") Insteel strands and a stirrup spacing of 75 mm (3") also reached the AASHTO nominal moment capacity for the section and failed by strand rupture.

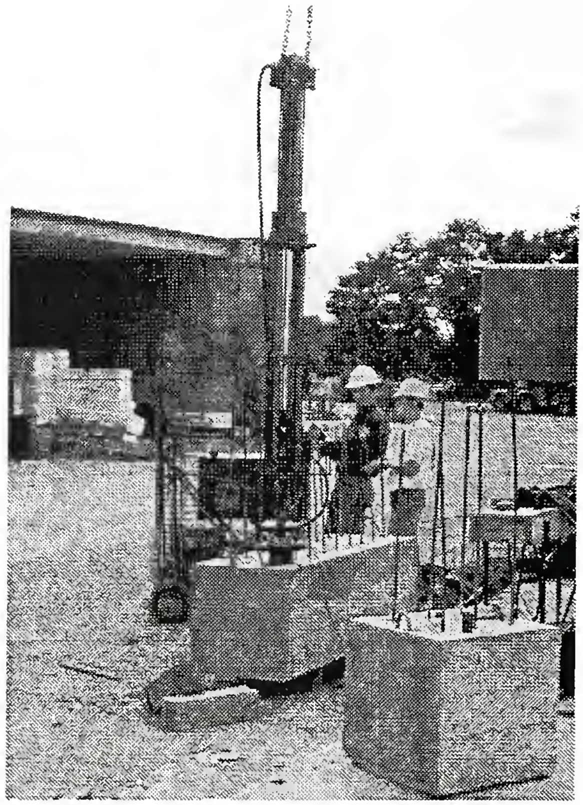


Figure 3.1-1 Testing of pullout specimen with 13.3 mm (1/2"-Special) strand.

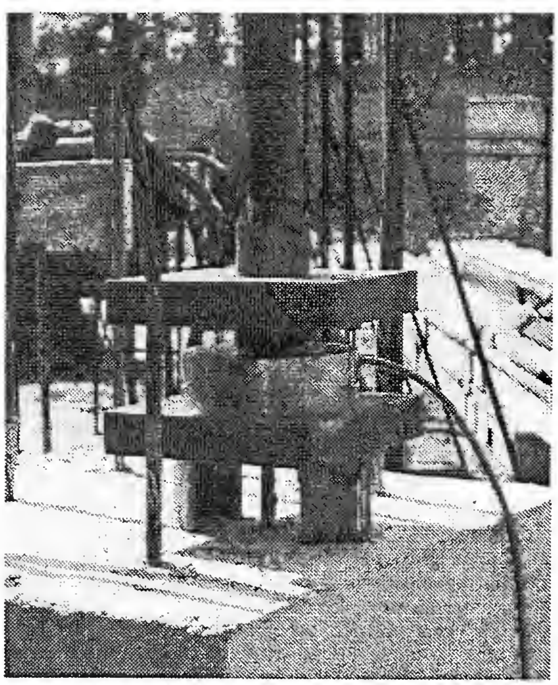


Figure 3.1-2 Load cell arrangement used to measure pullout force.

Pullout Specimen Summary
PCI Concrete Mix - Poured on 9/16/97

Pullout Tests Done On 9/18/97 (Concrete Compressive Strength was 33.1 MPa)

Insteel

Specimen	Pullout Load	First Slip
IST-9	162.1	139.2
IST-8	165.9	131.2
IST-7	182.4	142.3
IST-6	182.4	160.1
Average	173.2	143.2
Std. Dev.	9.25	10.59

Florida Wire & Cable

Specimen	Pullout Load	First Slip
FWC-9	171.0	92.5
FWC-8	169.0	72.1
FWC-7	172.6	67.2
FWC-6	168.1	85.0
Average	170.2	79.2
Std. Dev.	1.73	10.10

Pullout Tests Done On 9/20/97 (Concrete Compressive Strengh Was 35.9 MPa)

Insteel

Specimen	Pullout Load	First Slip
IST-5	175.0	150.3
IST-4	184.4	140.6
IST-3	192.2	163.2
IST-2	188.8	185.0
IST-1	182.8	165.5
Average	184.6	160.9
Std. Dev.	5.83	15.08

Florida Wire & Cable

Specimen	Pullout Load	First Slip
FWC-5	189.9	93.0
FWC-4	169.0	85.4
FWC-3	180.4	103.2
FWC-2	171.5	97.9
FWC-1	166.6	101.9
Average	175.5	96.3
Std. Dev.	8.60	6.49

* All loads are in kN
* All strand was 13.3 mm (1/2"-Special) 1860 MPa (270 ksi) Lo-Lax
* All Loads were obtained using a load cell except IST-6, where the load was based on the hydraulic pump pressure gage.

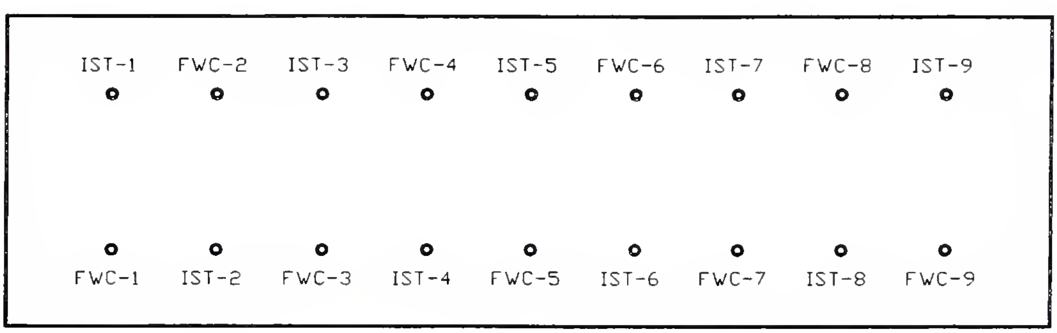


Figure 3.1-3 Data recorded for Pullout Specimen #1.

Pullout Specimen Summary
48 MPa (7000) psi NW Mix - Poured on 9/18/97

Pullout Tests Done On 9/20/97 (Concrete Compressive Strength was 58.6 MPa)

Insteel

Specimen	Pullout Load	First Slip
IST-10	137.4	120.1
IST-11	129.0	100.5
IST-12	151.9	112.3
IST-13	147.7	124.5
IST-14	129.4	101.6
IST-15	149.5	126.8
IST-16	143.7	114.3
IST-17	134.3	116.1
IST-18	134.8	117.9
Average	139.8	114.9
Std. Dev.	8.18	8.58

Florida Wire & Cable

Specimen	Pullout Load	First Slip
FWC-10	132.8	80.5
FWC-11	133.0	81.0
FWC-12	151.7	76.1
FWC-13	144.6	81.0
FWC-14	185.9	75.6
FWC-15	139.9	77.4
FWC-16	149.0	70.3
FWC-17	138.1	80.1
FWC-18	161.9	78.7
Average	148.5	77.8
Std. Dev.	15.92	3.29

* All loads are in kN
* All strand was 13.3 mm (1/2"-Special) 1860 MPa (270 ksi) Lo-Lax
* Specimen FWC-14 had a pull-out capacity that was considerably larger than that of all other specimens. This strand appeared to stop slipping after undergoing an initial slip of about 50 mm (2 inches). Load then increased before aditional slip occurred. The estimated slip at max. load was 90 mm (3 1/2) inches.

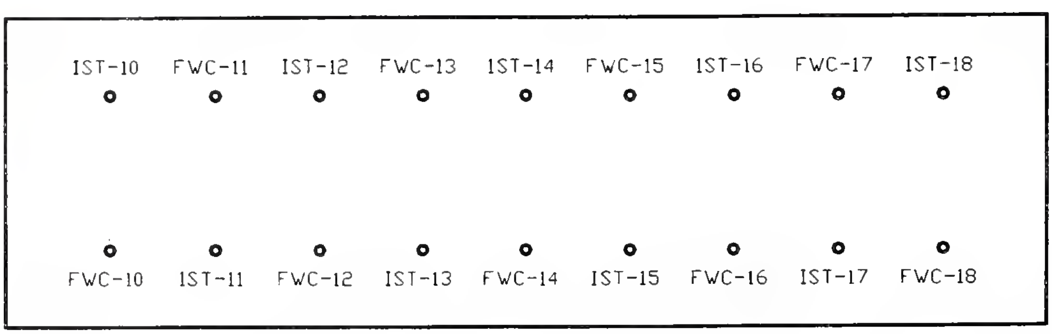


Figure 3.1-4 Data recorded for Pullout Specimen #2.

Pullout Specimen Summary
48 MPa (7000 psi) SLW Mix - Poured on 10/3/97

Pullout Tests Done On 10/7/97 (Concrete Compressive Strength was 48.3 MPa)

Insteel

Specimen	Pullout Load	First Slip
IST-27	122.3	101.6
IST-26	127.0	106.1
IST-25	139.7	111.0
IST-24	132.8	118.8
Average	130.5	109.4
Std. Dev.	6.49	6.36

Florida Wire & Cable

Specimen	Pullout Load	First Slip
FWC-27	153.5	77.6
FWC-26	143.2	78.5
FWC-25	138.3	71.2
FWC-24	124.3	80.1
Average	139.8	76.9
Std. Dev.	10.50	3.38

Pullout Tests Done On 2/28/98 (Concrete Compressive Strength was 61.4 MPa)

Insteel

Specimen	Pullout Load	First Slip
IST-23	177.1	139.0
IST-22	163.9	132.0
IST-21	171.6	136.8
IST-20	179.9	146.0
IST-19	166.5	120.1
Average	171.8	134.8
Std. Dev.	6.05	8.58

Florida Wire & Cable

Specimen	Pullout Load	First Slip
FWC-23	171.5	90.9
FWC-22	163.4	86.3
FWC-21	172.0	86.5
FWC-20	160.5	87.5
FWC-19	152.2	83.2
Average	163.9	86.9
Std. Dev.	7.38	2.49

* All loads are in kN
* All strand was 13.3 mm (1/2"-Special) 1860 MPa (270 ksi) Lo-Lax

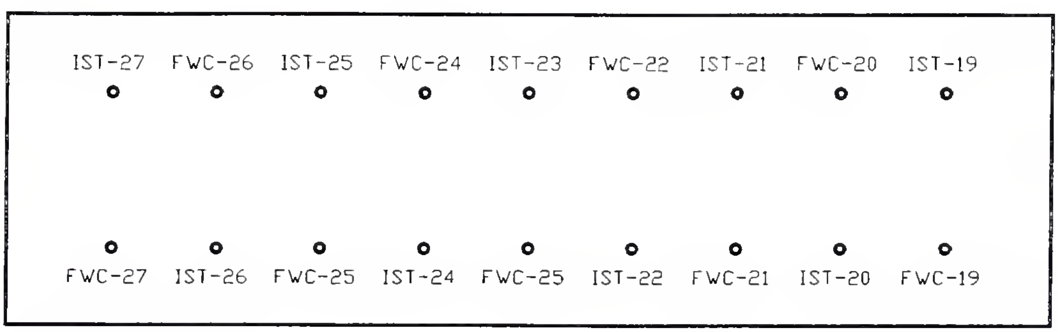


Figure 3.1-5 Data recorded for Pullout Specimen #3.

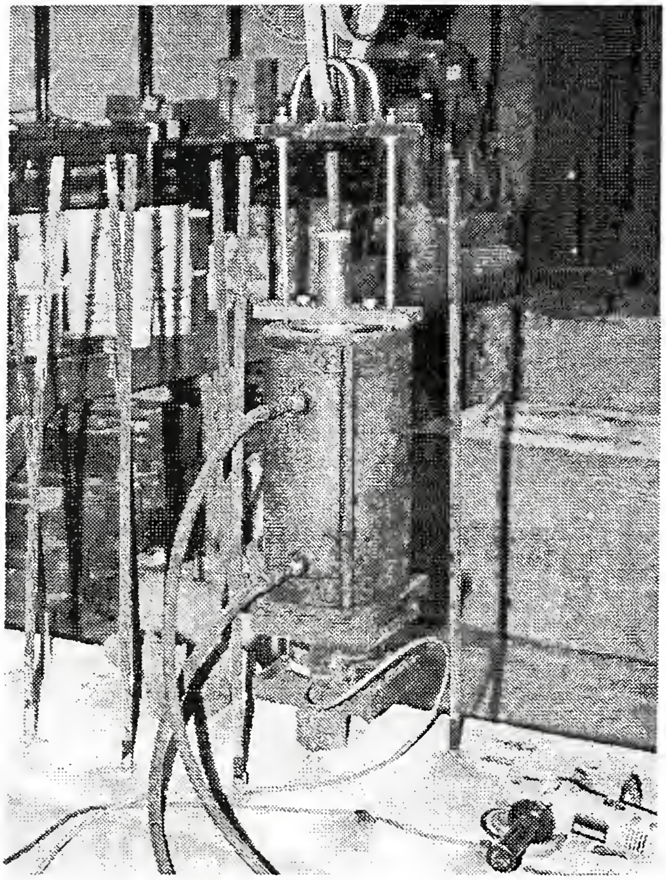


Figure 3.1-6 Testing of pullout specimen with 15.2 mm (0.6") strand.

Pullout Specimen Summary
69 MPa (10,000 psi) SLW Mix - Poured on 10/28/97

'ullout Tests Done On 6/18/98 and 6/19/98 (Concrete Compressive Strength was 96+ MPa

Insteel 13.3 mm (1/2"-Special) Strand

Specimen	Pullout Load	First Slip
IST-28	192.8	119.0
IST-29	192.2	74.9
IST-30	201.9	77.8
IST-31	190.4	129.4
IST-32	197.0	160.1
IST-33	183.5	112.8
IST-34	188.8	133.7
IST-35	182.4	105.6
IST-36	191.9	111.9
Average	191.2	113.9
Std. Dev.	5.74	25.13

Insteel 15.2 mm (0.6") Strand

Specimen	Pullout Load	First Slip
0.6-#1	249.8	197.0
0.6-#2	241.3	155.5
0.6-#3	227.5	164.4
0.6-#4	245.3	187.5
0.6-#5	258.0	110.8
0.6-#6	254.0	140.1
0.6-#7	244.4	201.0
0.6-#8	263.8	117.9
0.6-#9	260.0	****
Average	249.4	159.3
Std. Dev.	10.54	32.47

* All loads are in kN

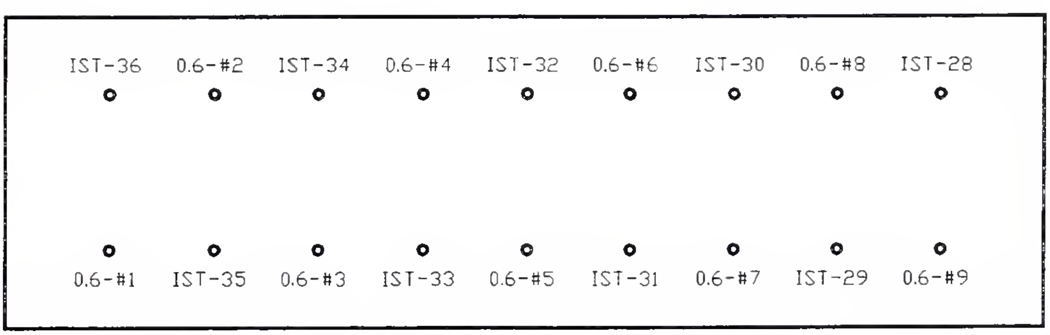


Figure 3.1-7 Data recorded for Pullout Specimen #4.

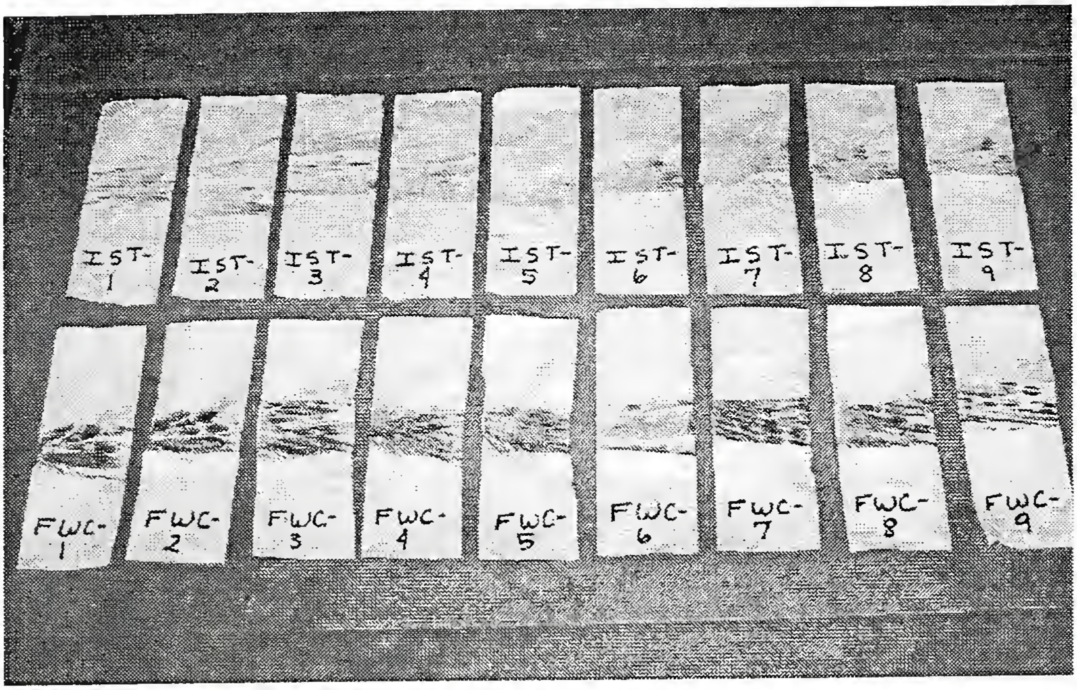


Figure 3.2-1 Towels used to wipe strands before placing them in Pullout Specimen #1.

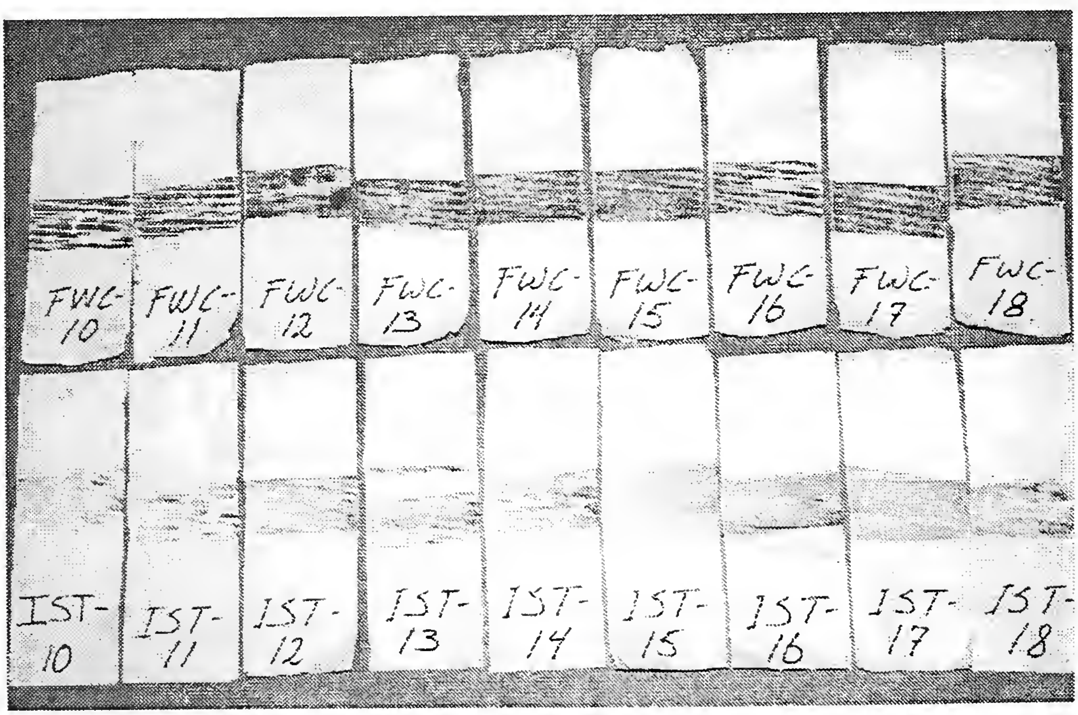


Figure 3.2-2 Towels used to wipe strands before placing them in Pullout Specimen #2.

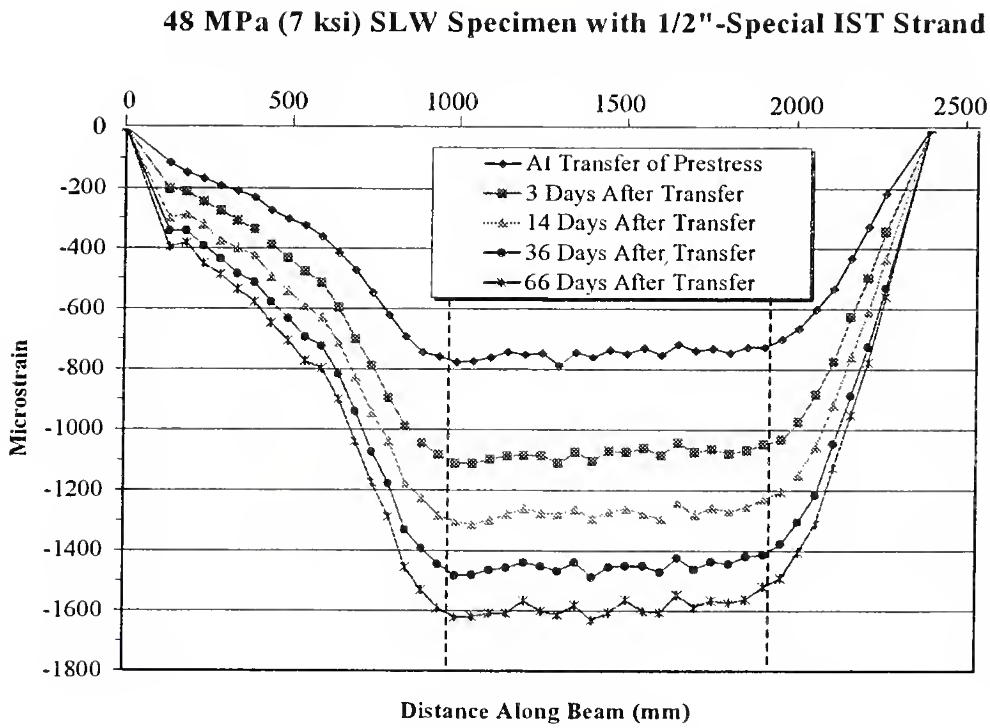


Figure 3.3-1 Surface strains for 13.3 mm IST strand in 48MPa SLW concrete.

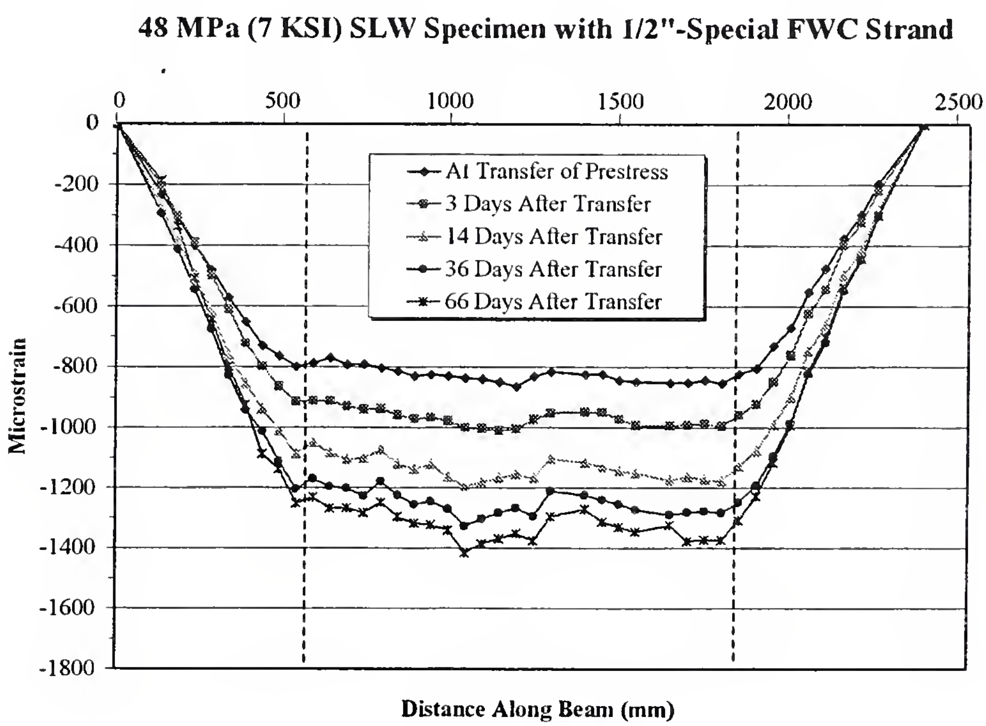


Figure 3.3-2 Surface strains for 13.3 mm FWC strand in 48MPa SLW concrete.

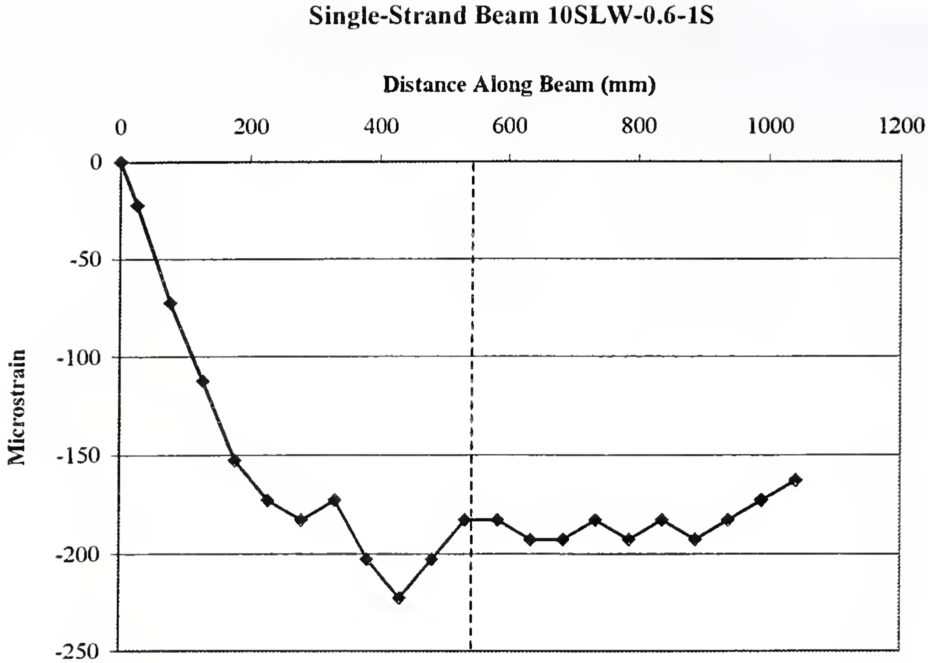


Figure 3.3-3 Surface strains for 15.2 mm IST strand in 69 MPa SLW concrete.

48 MPa (7,000 psi) Semi-Lightweight Beams w/ 13.3 mm (1/2”-Special) Strands

Florida Wire & Cable (7SLW-FWC)

Beam	Max. Load (kN)	Max. Moment (kN-m)	Deflection @ Max. Load (mm)	Failure Mode
1S	48.93	55.5	56.1	Flexure - Strand Rupture
1L	47.48	55.5	>76.0	Flexure - Strand Rupture
2S	48.84	55.3	44.0	Flexure - Strand Rupture
2L	46.83	54.8	64.9	Shear, then Strand Rupture
3S	49.73	56.3	51.5	Flexure - Strand Rupture
3L	46.02	53.8	74.1	Flexure - Strand Rupture

Insteel (7SLW-IST)

Beam	Max. Load (kN)	Max. Moment (kN-m)	Deflection @ Max. Load (mm)	Failure Mode
1S	47.78	54.2	53.0	Shear
1L	46.12	54.0	>76.0	Shear
2S	47.17	53.6	69.6	Flexure - Strand Rupture
2L	44.51	52.2	78.5	Shear, then Strand Rupture
3S	46.89	53.1	65.5	Flexure - Strand Rupture
3L	46.06	54.0	61.4	Flexure - Strand Rupture

AASHTO Nominal Moment Capacity = 46.9 kN-m (f_{ps} = 1793 MPa)

Strain Compatibility ...

- $\varepsilon_c = 0.004$
- Concrete Stress-Strain Relationship Represented By Parabola
- Strand Stress-Strain Relationship Represented By Tadros Curve

Beams 1 & 2 (f'c= 69.0 MPa $\Rightarrow$ Mn = 53.0 MPa (f_{ps} = 2006 MPa)
Beam 3 (f'c= 62.0 MPa $\Rightarrow$ Mn = 52.2 MPa (f_{ps} = 1979 MPa)

Figure 3.4-1 Test data for the 48 MPa (7 ksi) single-strand beams in metric units.

7,000 psi Semi-Lightweight Beams w/ 1/2"-Special Strands

Florida Wire & Cable (7SLW-FWC)

Beam	Max. Load (lb)	Max. Moment (kip-ft)	Deflection @ Max Load (inches)	Failure Mode
1S	11,000	40.9	2.2	Flexure - Strand Rupture
1L	10,680	40.9	>3.0	Flexure - Strand Rupture
2S	10,980	40.8	1.7	Flexure - Strand Rupture
2L	10,530	40.4	2.6	Shear, then Strand Rupture
3S	11,180	41.5	2.0	Flexure - Strand Rupture
3L	10,350	39.7	2.9	Flexure - Strand Rupture

Insteel (7SLW-IST)

Beam	Max. Load (lb)	Max. Moment (kip-ft)	Deflection @ Max Load (inches)	Failure Mode
1S	10,740	40.0	2.1	Shear
1L	10,370	39.8	3.1	Shear
2S	10,600	39.5	2.7	Flexure - Strand Rupture
2L	10,000	38.5	>3.0	Shear, then Strand Rupture
3S	10,540	39.2	2.6	Flexure - Strand Rupture
3L	10,350	39.8	2.4	Flexure - Strand Rupture

AASHTO Nominal Moment Capacity = 34.6 kip-ft (f_{ps} = 260 ksi)

Strain Compatibility ...

- $\varepsilon_c = 0.004$
- Concrete Stress-Strain Relationship Represented By Parabola
- Strand Stress-Strain Relationship Represented By Tadros Curve

Beams 1 & 2 (f'c=10 ksi ⇒ Mn = 39.1 kip-ft (f_{ps} = 291 ksi)
Beam 3 (f'c= 9 ksi ⇒ Mn = 38.5 kip-ft (f_{ps} = 287 ksi)

Figure 3.4-2 Test data for the 48 MPa (7 ksi) single-strand beams in U.S. customary units.

Insteel (10SLW-0.6) Metric Units

Beam	Max. Load (kN)	Max. Moment (kN-m)	Deflection @ Max Load (mm)	Failure Mode
1S	53.42	67.9	63.8	Flexure - Strand Rupture
1L	52.78	68.5	>101.9	Shear
2S	56.38	68.9	59.5	Shear
2L	53.55	68.5	79.6	Shear
3S	56.58	65.8	77.3	Flexure - Strand Rupture
3L	54.17	68.2	101.1	Flexure - Strand Rupture

AASHTO Nominal Moment Capacity = 61.1 kN-m (f_{ps} = 1795 MPa)

Strain Compatibility ... Mn = 67.9 kN-m (f_{ps} = 1980 MPa)

- f'c = 83 MPa
- ε_c = 0.004
- Concrete Stress-Strain Relationship Represented By Parabola
- Strand Stress-Strain Relationship Represented By Tadros Curve

Insteel (10SLW-0.6) U.S. Customary Units

Beam	Max. Load (lb)	Max. Moment (kip-ft)	Deflection @ Max Load (in)	Failure Mode
1S	12,010	50.1	2.5	Flexure - Strand Rupture
1L	11,860	50.5	>4.0	Shear
2S	12,670	50.8	2.3	Shear
2L	12,015	50.5	3.1	Shear
3S	12,710	48.5	3.0	Flexure - Strand Rupture
3L	12,170	50.3	4.0	Flexure - Strand Rupture

AASHTO Nominal Moment Capacity = 45.1 kip-ft (f_{ps} = 260 ksi)

Strain Compatibility ... Mn = 50.1 kip-ft (f_{ps} = 287 ksi)

- f'c = 12 ksi
- ε_c = 0.004
- Concrete Stress-Strain Relationship Represented By Parabola
- Strand Stress-Strain Relationship Represented By Tadros Curve

Figure 3.4-3 Test data for the 69 MPa (10,000 psi) single-strand beams.

48 MPa (7000 psi) T-Beams w/ 13.3 mm (1/2”-Special) Strands

Metric

Beam	Max. Load (kN)	Specimen Age (Days)	Interpolated Conc. Cylinder Strength (MPa)	Max. Moment (kN-m)	Deflection @ Max Load (mm)	Failure Mode
IST1	536.1	14	58	492.0	51.2	Flexure – Strand Rupture
IST2	551.7	25	57	505.8	36.1	Flexure – Strand Rupture
FWC	536.2	21	59	492.0	20.8	Bond
FWC-3”	577.7	20	57	529.4	35.2	Flexure – Strand Rupture
FWC-6”	489.2	17	55	450.1	7.9	Bond / Web Shear Failure
FWC-15”	444.9	27	49	410.3	6.2	Bond / Web Shear Failure

AASHTO Nominal Moment Capacity (Mn) = 461.4 kN-m (f_{ps} = 1827 MPa)

Strain Compatibility ... Mn = 491.9 kN-m (f_{ps} = 1945 MPa)

U.S. Customary Units

Beam	Max. Load (kips)	Specimen Age (Days)	Interpolated Conc. Cylinder Strength (psi)	Max. Moment (kip-ft)	Deflection @ Max Load (in.)	Failure Mode
IST1	120.5	14	8460	362.9	2.02	Flexure – Strand Rupture
IST2	124.0	25	8300	373.1	1.42	Flexure – Strand Rupture
FWC	120.5	21	8600	362.9	0.82	Bond
FWC-3”	129.9	20	8200	390.5	1.39	Flexure – Strand Rupture
FWC-6”	110.0	17	8000	332.0	0.31	Bond / Web Shear Failure
FWC-15”	100.0	27	7100	302.6	0.24	Bond / Web Shear Failure

AASHTO Nominal Moment Capacity (Mn) = 340.3 kip-ft (f_{ps} = 265 ksi)

Strain Compatibility ... Mn = 362.8 kip-ft (f_{ps} = 282 MPa)

Figure 3.5-1 Test data for the 48 MPa (7,000 psi) T- beams.

69 MPa (10,000 psi) T-Beams w/ 15.2 mm (0.6") Strands

Metric

Max. Load (kN)	Specimen Age (Days)	Concrete Cylinder Strength (MPa)	Max. Moment (kN-m)	Deflection @ Max Load (mm)	Failure Mode
622.8	14	68.6	660.6	46.6	Flexure – Strand Rupture

AASHTO Nominal Moment Capacity (Mn) = 601.4 kN-m (f_{ps} = 1827 MPa)

Strain Compatibility ... Mn = 638.8 kN-m (f_{ps} = 1937 MPa)

U.S. Customary Units

Max. Load (kips)	Specimen Age (Days)	Concrete Cylinder Strength (psi)	Max. Moment (kip-ft)	Deflection @ Max Load (mm)	Failure Mode
140.0	14	9950	487.2	1.83	Flexure – Strand Rupture

AASHTO Nominal Moment Capacity (Mn) = 443.6 kip-ft (f_{ps} = 265 ksi)

Strain Compatibility ... Mn = 471.2 kip-ft (f_{ps} = 281 ksi)

Figure 3.5-2 Test data for the 69 MPa (10,000 psi) T- beam.

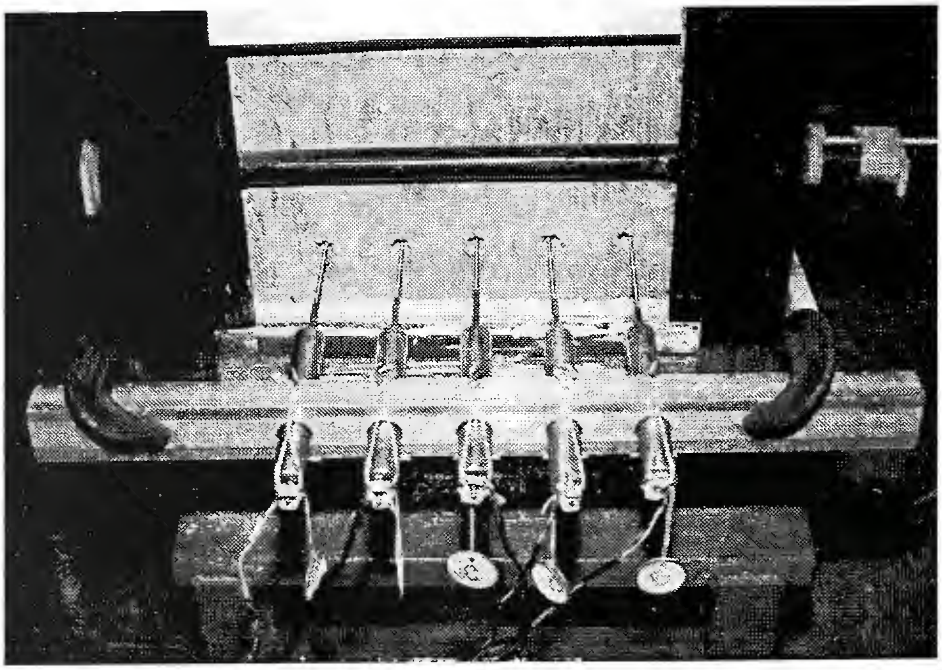


Figure 3.5-3 Strands pulled in from the ends of T-Beam FWC.

Load (kips)	Strand Slip in Inches A	B	C	D	E
120.12	0.002	0.002	0.005	0.001	0.000
120.15	0.002	0.002	0.005	0.001	0.000
120.53	0.002	0.002	0.005	0.001	0.000
120.54	0.002	0.002	0.005	0.001	0.000
120.54	0.002	0.002	0.005	0.001	0.000
120.54	0.002	0.002	0.006	0.001	0.000
120.52	0.002	0.003	0.008	0.001	0.000
120.51	0.002	0.004	0.011	0.001	0.000
120.51	0.002	0.004	0.016	0.002	0.000
120.50	0.002	0.004	0.021	0.002	0.000
120.49	0.002	0.005	0.026	0.003	0.000
120.51	0.002	0.005	0.031	0.003	0.000
120.51	0.002	0.005	0.033	0.003	0.000
120.52	0.002	0.005	0.036	0.003	0.000
120.52	0.002	0.005	0.037	0.003	0.000
120.52	0.002	0.005	0.040	0.003	0.000
120.51	0.002	0.005	0.045	0.003	0.000
120.48	0.002	0.005	0.051	0.004	0.000
119.03	0.004	0.045	0.116	0.064	0.001
118.72	0.007	0.156	0.230	0.186	0.002
118.71	0.008	0.196	0.270	0.226	0.003
117.48	0.011	0.296	0.367	0.325	0.004
107.66	0.200	0.559	0.615	0.569	0.175

Figure 3.5-4 Strand-slip data for T-Beam FWC.

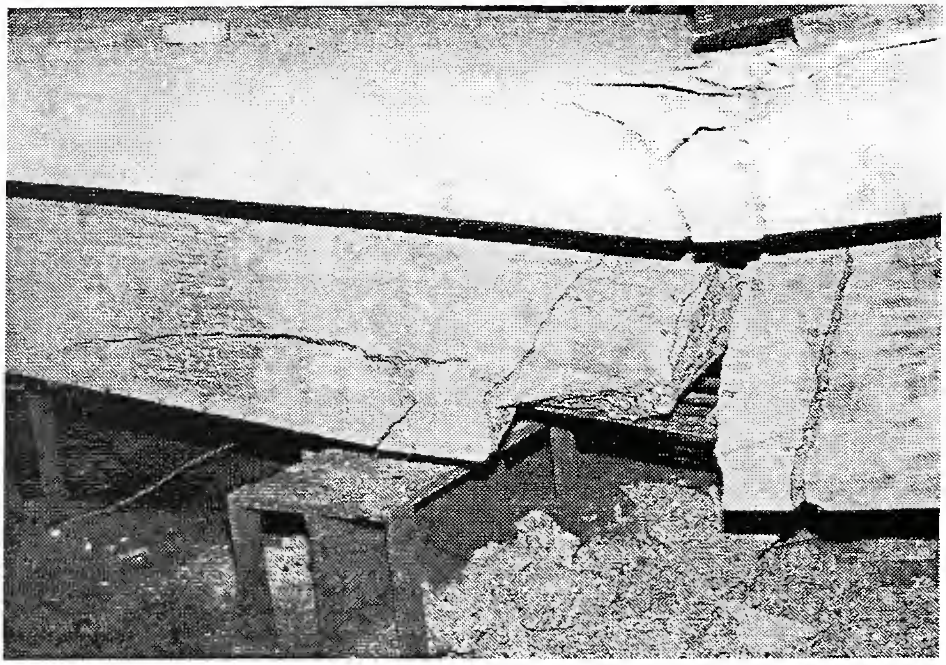


Figure 3.5-5 Flexure-shear cracking, and subsequent splitting in T-Beam FWC.

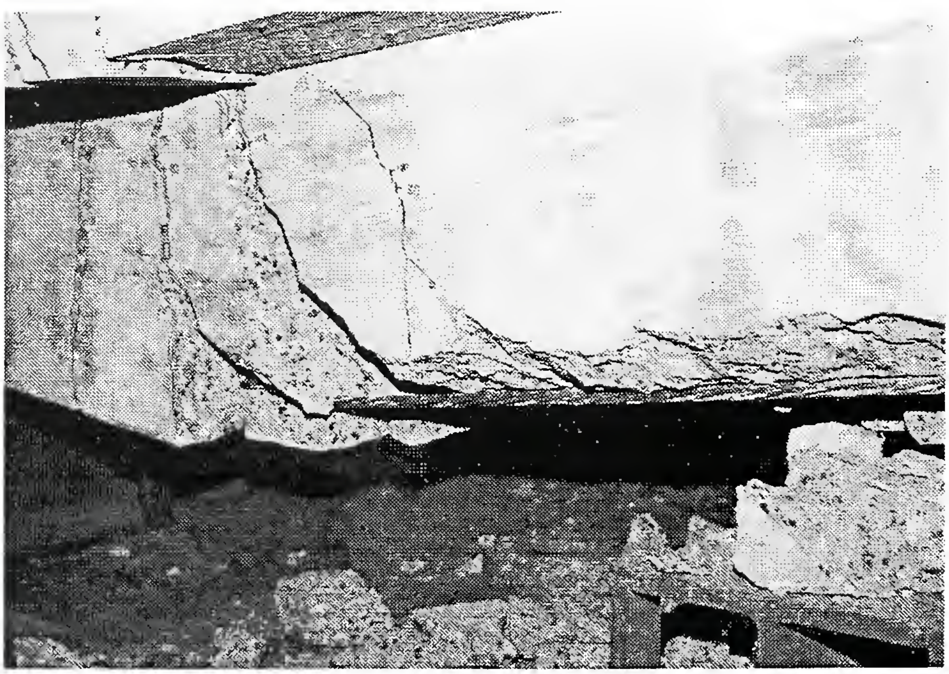


Figure 3.5-6 Exposed strand associated with bond failure in T-Beam FWC.

Figure 3.5-7 Failure of T-Beam FWC-6".

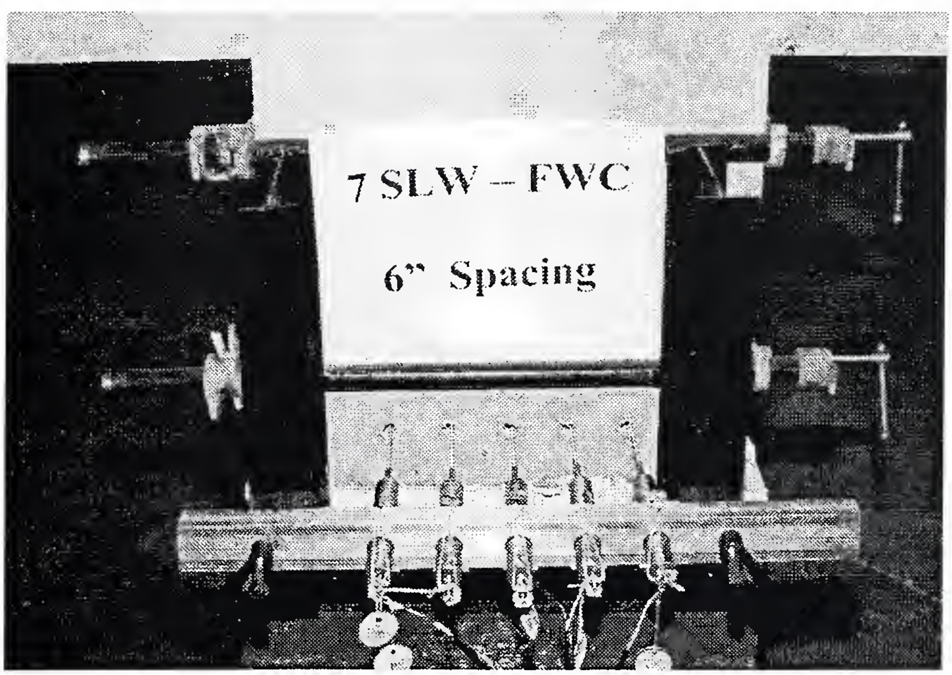


Figure 3.5-8 Figure showing strand slip in T-Beam FWC-6".

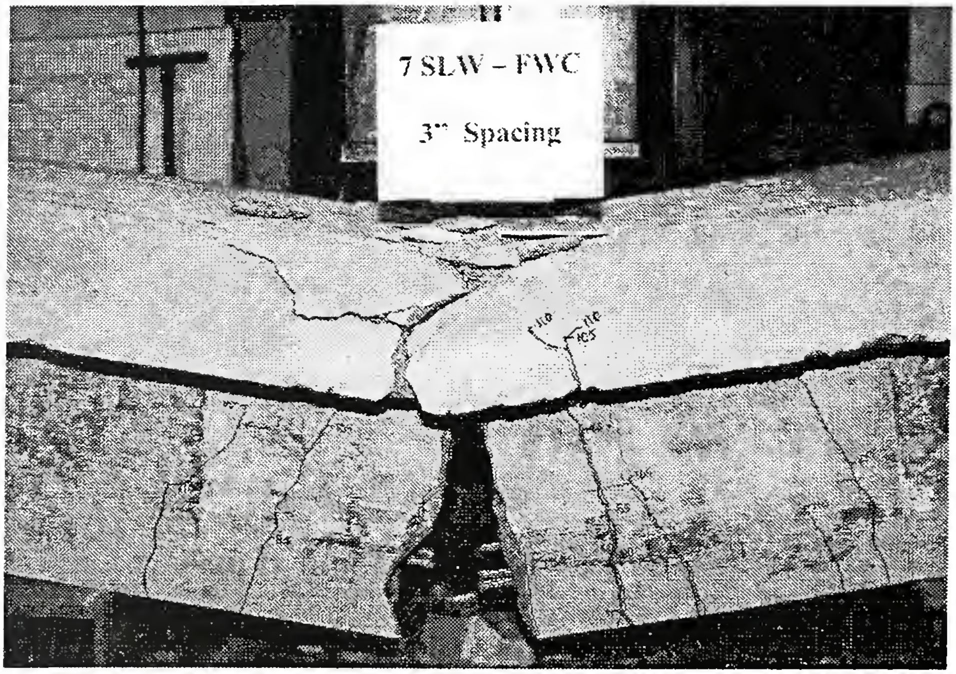


Figure 3.5-9 Failure occurred by strand rupture in T-Beam FWC-3".

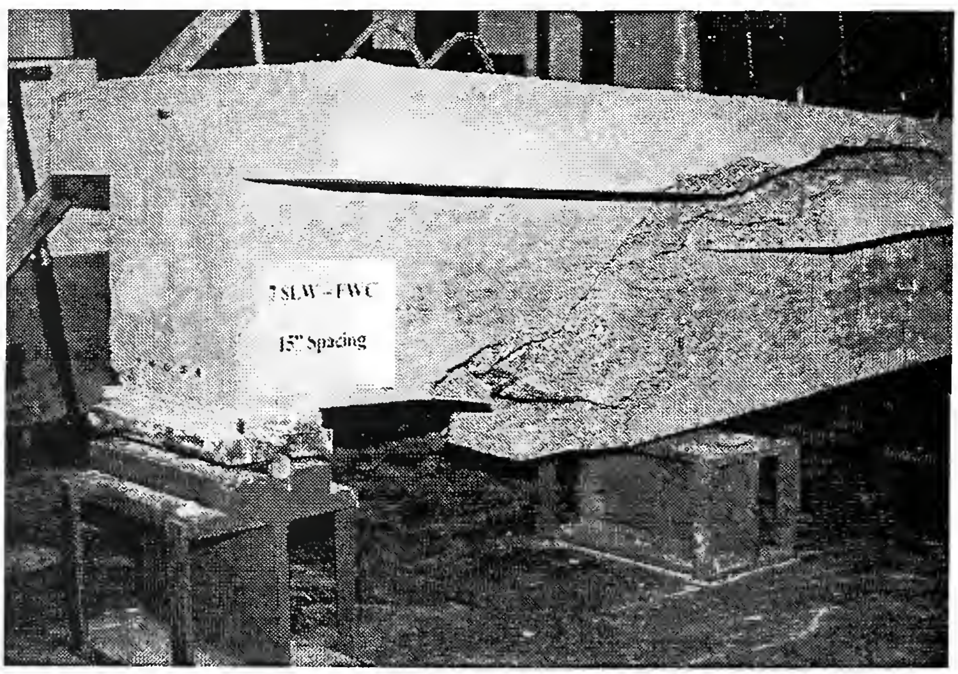


Figure 3.5-10 Failure of T-Beam FWC-15"

CHAPTER 4 - CONCLUSIONS AND RECOMMENDATIONS

4.1 Discussion of Results

The findings from this study support the notion that there is an interaction between the shear carried by a prestressed member near the point of maximum moment and the length required to sufficiently anchor the longitudinal reinforcement. Although the findings of this study were made in the context of tests on members with semi-lightweight concrete, which typically have a lower modulus of rupture and would thus be more susceptible to flexure-shear cracking, the principles discussed herein should also be applicable for members cast with normal-weight concrete.

Tests on single-stranded rectangular beams and multiple-stranded T-Beams revealed that the length required to develop the tensile capacity of a strand in concrete is, in some cases, dependent on the member geometry and loading configuration. All combinations of strand and concrete mixes used in the single-strand rectangular-beam specimens this study resulted in failure flexural capacities greater that the calculated ones using the 16th Edition of the AASTO Standard Specifications.

When the same combinations were tested in the multi-stranded T-Beams, however, the results were mixed. The combination of 13.3 mm (1/2"-Special) strand produced by Florida Wire & Cable and a 48MPa (7000 psi) concrete mix resulted in bond failures for three of the four T-Beam specimens tested at failure loads below the calculated ones using the AASHTO Specifications. Two other T-Beam specimens, which had the same concrete mix and the same diameter strand supplied by Insteel, resulted in flexural failures by strand rupture at loads greater than the calculated ones using the AASHTO Specifications.

The investigators noted, upon review of a videotaped failure of T-Beam FWC in this study, that the bond failure was preceded by a flexure-shear crack. This observation led to the hypothesis that the onset of cracking shifted the maximum tensile stress (i.e. the "critical" section) in the strand from the point of maximum moment towards the end of the beam. This created a region of constant tensile stress demand between the section of maximum moment and the new "critical" section. Although similar crack patterns were noted for the T-Beams containing the 13.3 mm (1/2"-Special) strand produced by Insteel, these specimens failed by strand rupture. It was surmised that the actual development length for the Insteel strand may have been considerably less than the calculated specification value as tested. If this were the case, then a shift in the "critical" section would result in an embedment length to the critical section that was still larger than the actual development length for the strand, and collapse would not occur. If the actual development length of the Florida Wire & Cable strand was close to the specification-determined value as tested, then a shift in the critical section could lead to collapse.

To test this theory, three additional T-Beam specimens containing the same mix and strand combination used in first T-Beam FWC were fabricated and tested. However, the additional T-Beam specimens each had different amounts of stirrup reinforcement, 76 mm (3 inches), 152 mm (6 inches), and 381 mm (15 inches). For the T-Beam with the closest stirrup spacing at the point of maximum moment, namely T-Beam FWC-3", stirrup spacing of 76 mm (3 inches), bond failure was prevented (presumably by minimizing the shift in the location of the critical section) and the mode of failure was flexure by strand rupture. The amount of transverse reinforcement required to prevent bond failure was between 2.5 to 5 times the amount required by shear design (Note: T-Beam FWC-6”, spacing of 152 mm, failed by bond while T-Beam FWC-3” failed by strand rupture). While this may not be practical for most design situations, one must consider that the critical section may shift considerably in the event of diagonal cracking. This will be further discussed in Section 4.3.

4.2 Conclusions

Based on the work carried out in this study, the following conclusions are drawn.

1. Both of the 13.3 mm (1/2”-Special) strands used in this study met the requirement for the minimum average pullout force to exceed 167 kN (37.6 kips) when testing according to the Moustafa procedure, thereby indicating that the initial bond quality of the strands was acceptable.
2. Additional Moustafa pullout tests conducted with strand specimens from the same reels, but cast with different concrete mixes, resulted in significantly lower pullout capacities. This was true even when the additional pullout blocks had a considerably higher concrete compressive strength at the time of testing. Therefore, it is evident that concrete parameters, other than strength, can greatly affect the results of Mustafa test.
3. Measurements of concrete surface strains indicated that the transfer lengths associated with all combinations of strand and semi-lightweight (SLW) concrete evaluated in this study were less than code assumed 50 strand diameters in the absence of longitudinal splitting near the ends. In the presence of splitting cracks, the measured transfer length increased to almost 70 strand diameters. Transfer-length measurements for the 48 MPa (7000 psi) concrete indicated that the transfer lengths remained essentially unchanged during the first 60 days following transfer of the prestressing force.

4. Eighteen load tests on rectangular single-strand beams indicated that the AASHTO and ACI development lengths of $L_d = \left(f_{ps} - \frac{2}{3}f_{se}\right)d_b$ provided sufficient embedment to develop their calculated moment capacity with the semi-lightweight concrete mixes used. In eight of the twelve 48 MPa single-strand beam end tests, the strands ruptured at failure. In the other four a shear failure occurred after significant amount of deflection.
5. Tests on 48 MPa (7000 psi) T-Beams with multiple strands resulted in flexural failures by strand rupture for the two specimens containing 13.3 mm (1/2"-Special) Insteel strand. A similar test, conducted on a T-Beam with concrete from the same batch and 13.3 mm (1/2"-Special) strands produced by Florida Wire & Cable, resulted in bond failure after the occurrence of a flexure-shear crack. Transverse reinforcement consisted of 13 mm (#4) stirrups at 152 mm (6") on center for the entire length of all three T-Beams. Similar combinations of 13.3 mm (1/2"-Special) Florida Wire & Cable strand and 48 MPa (7000 psi) SLW concrete resulted in strand rupture in the single-strand beam tests.
6. Development-length tests on two additional T-Beams also resulted in bond failures for the combination of 13.3 mm Florida Wire & Cable strands and 48 MPa SLW concrete. Transverse reinforcement consisted of 13 mm (#4) stirrups at spacings of 152 mm and 381 mm, respectively, in the central portion of these additional simply-supported specimens. The minimum amount of transverse reinforcement required by AASHTO shear design of these specimens was 13mm (#4) stirrups at 381 mm (15") on center.
7. Flexural failure, by strand rupture, occurred for a third additional T-beam specimen that utilized 13.3 mm Florida Wire & Cable strands and 48 MPa SLW concrete when 13 mm (#4) stirrups in the central portion of the simply-supported specimen were spaced at 75

mm (3”) on center. This spacing corresponded to five times the area of transverse reinforcement required for shear.

8. A T-Beam test utilizing 69 MPa SLW concrete and 15.2 mm (0.6”) Insteel strand resulted in flexural failure, by strand rupture, when loaded at a distance of L_d, based on the 16th Edition of the AASHTO Specifications, from each end. This specimen had 13 mm (#4) stirrups at 75 mm (3”) on center throughout the central portion of the simply-supported span.

4.3 Recommendations

The following recommendations are made based on the results of this study for the strand types and concrete mixes evaluated.

1. The current assumption for transfer length estimate of 50 strand diameters was conservative for all combinations of strand and concrete mixes tested in the absence of splitting cracks at the ends. In the end specimen where longitudinal splitting was observed, the transfer length measured was 70 strand diameters. Therefore, the estimate of 50 strand-diameters for transfer length can be used when checking shear provisions for prestressed members with semi-lightweight concrete, provided the splitting stresses do not exceed the tensile strength of the concrete. Otherwise, it is recommended to use a transfer length estimate of 70 strand diameters.
2. A shift in the location of the critical section may occur due to flexure-shear cracking. Therefore, it is recommended that the current requirements for development length be enforced at a section located a distance “d_p” from the critical section based on flexure

requirements in the direction of the its free end. In this check, d_p is the distance from the extreme compression fiber to the centroid of the prestressed reinforcement, but no less than 80% of the overall member height. This recommendation may appear to be too conservative at first glance. However, the implications for most design situations will be small. For shallow members, checking development length requirements at a relatively small distance of d_p will not be overtaxing on design. For larger depth members with fully bonded strands, the issue of development length is seldom, a critical factor in the design. It must be noted that all the multiple strand specimens in this study were designed so as to avoid web-shear cracking near the member ends. The presence of a shear crack near the member end, intercepting the transfer length of the strand, could result in the strands slipping prematurely.

APPENDIX A

PULL-OUT TEST PROCEDURE (MOUSTAFA METHOD)

Note: The material in this appendix originally appeared in March/April 1997 issue of the Journal of the Precast / Prestressed Concrete Institute (vol.42, no.2) as part of a special report authored by Donald R. Logan, P.E. titled "Acceptance Criteria for Bond Quality of Strand for Pretensioned Prestressed Concrete Applications." The material herein is reproduced with the permission of the Precast / Prestressed Concrete Institute and Mr. Logan.

APPENDIX A – PULL-OUT TEST PROCEDURE (MOUSTAFA METHOD)

OBJECTIVE

Determine the pull-out capacity of as-received strand samples (protected from weathering) and compare that pull-out capacity with the most recent benchmark established in Stresscon Corporation's bond test conducted in May-June 1996 (see Fig. E1). Four strand groups attained transfer and development lengths considerably shorter than the lengths computed by the ACI equations. The average pull-out capacities of each of these four groups ranged from 36.8 to 41.6 kips (164 to 185 kN), respectively.

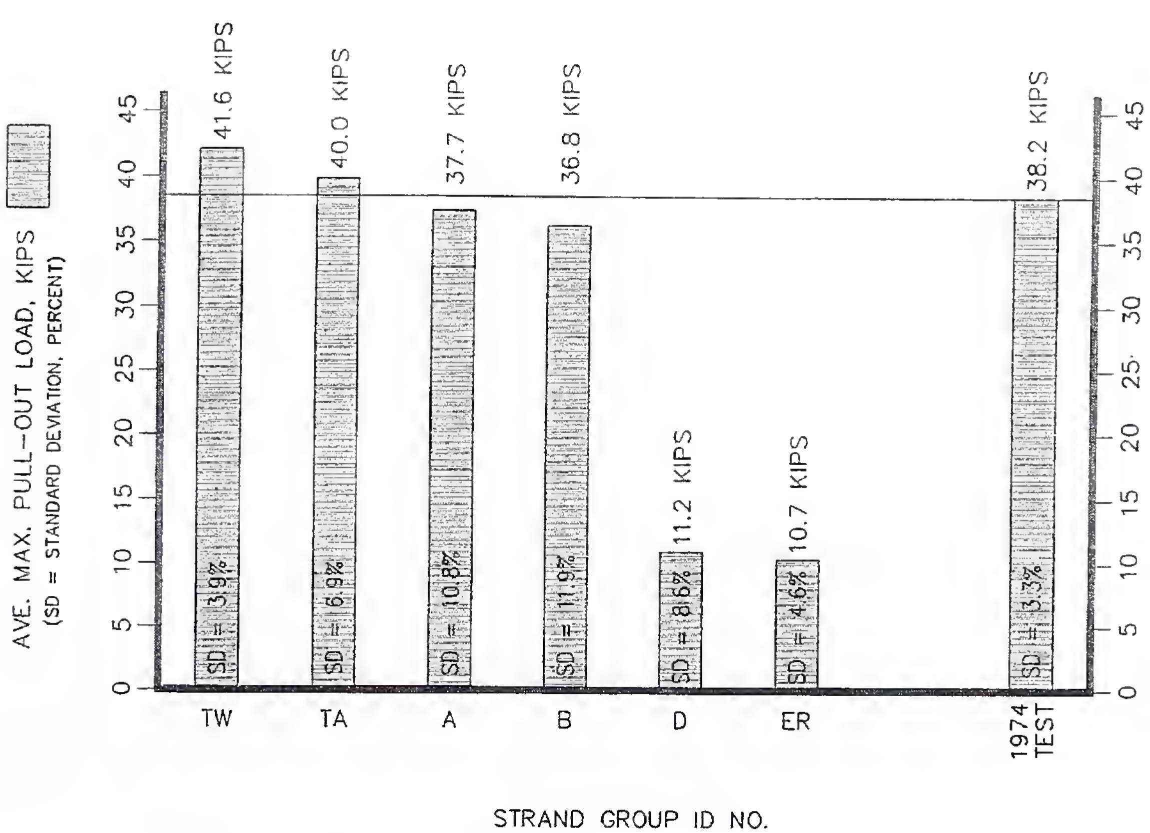
Based on the excellent transfer/development length performance of all of these top four strand groups, the following benchmark is recommended as the minimum acceptable pull-out capacity:

Average pull-out load = 36 kips (160 kN)
(set of six samples)
Maximum standard deviation = 10 percent

Note that this capacity is only applicable to 0.5 in. (13 mm) diameter, 270 ksi (1862 MPa) strand with an 18 in. (457 mm) embedment, cast in normal weight, well vibrated concrete having a concrete strength at the time of the pull-out test between 3500 and 5900 psi (24.1 and 40.7 MPa).

GENERAL PROCEDURAL COMMENT

To attain results consistent with a long series of tests extending back to 1974, it is of primary importance to closely follow the procedure used in the 1974 and 1992 tests conducted at Concrete Technology Corporation, Tacoma, Washington, and an extensive series of tests subsequently conducted at Stresscon Corporation, Colorado Springs, Colorado, since 1992. This procedure was first developed by Saad Moustafa in 1974 and was modified by Donald Logan, who introduced the 2 in. (51 mm) sleeve at the top concrete surface to eliminate the effects of surface spalling, and es-



Notes:
(1) The above results are from the strand bond tests conducted at Stresscon Corporation and supervised by Saad Moustafa in May 1996 (six specimens per test).
(2) Strand specimens were embedded 18 in. (457 mm) into well vibrated concrete test blocks. Concrete was Stresscon's standard production mix. All strand was 0.5 in. (13 mm) in diameter.
(3) All strand specimens, except TW, were in their as received condition and were protected from weathering.

tablished the 20 kips per minute (89 kN/minute) load application rate, which is close to the average rate observed in earlier tests.

STRAND PREPARATION PROCEDURE

1. Six strand samples shall be taken from a fresh, unopened pack of unweathered strand (as-received from the manufacturer and not modified in any way by the manufacturer). Samples are to be saw-cut to 34 in. (864 mm) lengths, any projections from the saw-cutting will be removed, and the samples will be straightened by hand if they are bowed more than $^3/_8$ in. (9.5 mm) in their 34 in. (864 mm) length.

2. The strand samples shall be visually examined to verify that they are not rusted. They shall be wiped with a clean paper towel to clean off any loose dirt or incidental rust and to observe the residue on the strand as received from the strand manufacturer. The samples shall *not* be cleaned with acid or any other solvent.

3. If more than one shipment of strand (or more than one manufacturer's strand) is being tested for comparative performance, duct-tape tags shall be attached to the top end of all samples in accordance with an identification system. Each tag shall be marked with indelible ink with its appropriate symbol, and taped securely in a location where they will be visible after casting of the test block.

4. The taped samples shall be tied securely in each test block at the locations indicated in the test block layout drawing. If more than one group is being tested, it is important to have each test block contain an equal number of strand samples from each group distributed alternately throughout that block. This will ensure that each group receives equal concrete quality and equal placement and vibration of the concrete. Refer to Fig. E2 for an example of a test using three different strand groups.

CASTING PROCEDURE

1. Test block forms shall be set up, reinforcing cages installed and securely positioned before any strand samples are tied in place.

2. After the forms and reinforcement have been checked, the tagged strand samples shall be tied securely in place in accordance with the layout shown in the test block layout drawing. The time that the strands are exposed to the weather shall be minimized.

3. Immediately after the strand location and tying procedure is checked and approved, concrete placement shall take place.

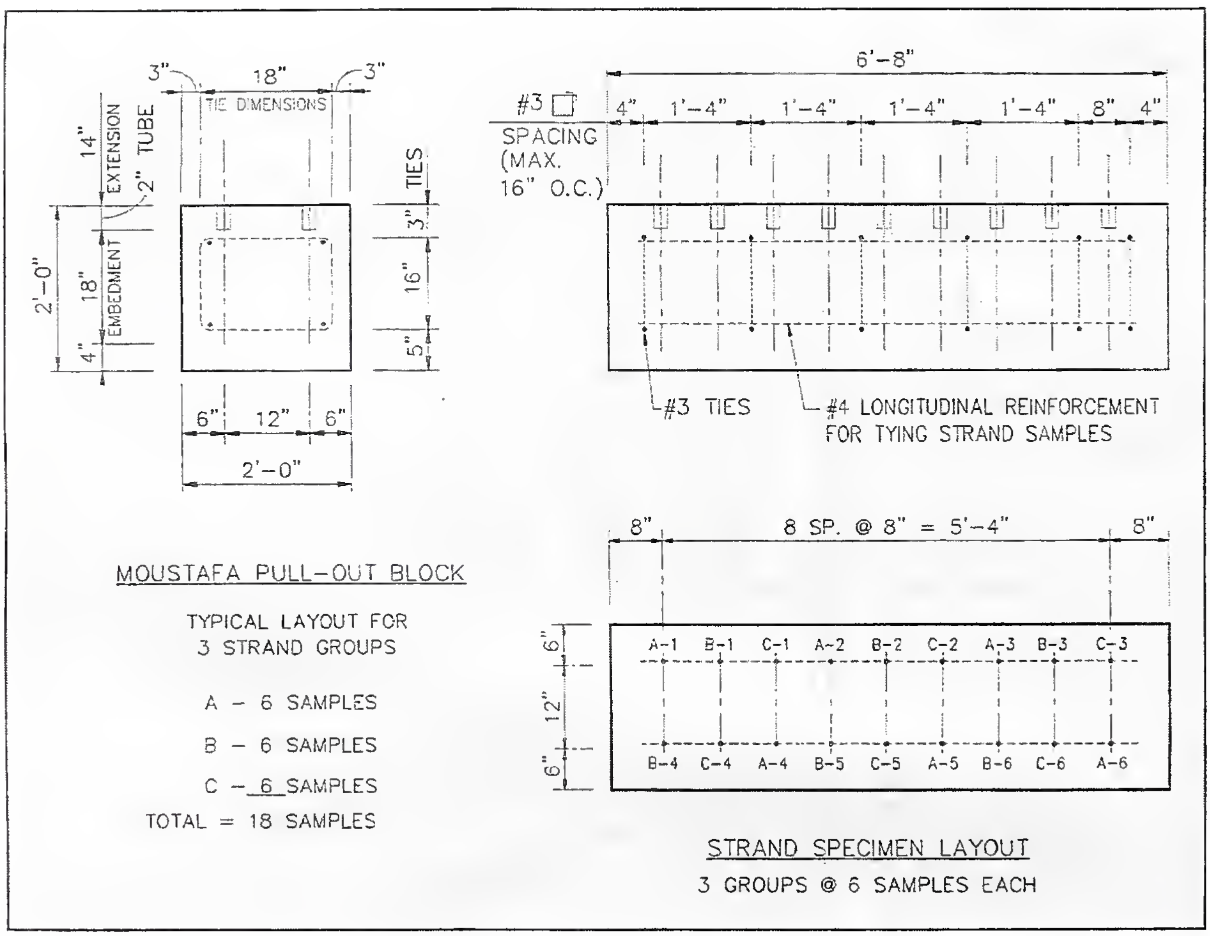


Fig. E2. Details of pull-out test block (Moustafa method).

Table E1. Suggested concrete mix design.

Materials	Quantity per cubic yard*
Cement (Type III)	660 lbs (299 kg)
Concrete sand	1100 lbs (499 kg) (SSD)
Crushed gravel [3/4 in. (19 mm)]	1900 lbs (862 kg)
Normal range water reducer	26 oz. (737 g)
Air-entraining agent	0 oz.
High range water reducer	0 oz.
Water	35 gal. (132 l)

* 1 cubic yard = 0.7646 m³

4. The concrete will be produced from one batch of hard-rock structural concrete mix (without any high range water reducers) that is expected to attain between 3800 and 5000 psi (26.2 and 34.5 MPa) with overnight heat curing (or 2 days of ambient cure). Four cylinders shall be cast from that batch and cured with the test blocks to determine the concrete strength at the time of the test (three cylinders) and one cylinder saved for a 28-day test. A suggested concrete mix design is shown in Table E1.

5. The concrete shall be well-vibrated using internal vibrators, with the concrete at approximately 3 in. (76 mm) slump. The intent of the vibration is to duplicate good, production quality consolidation around the strand samples.

6. The top surface shall be smoothed using a one-pass trowel finish in order to attain flat concrete surfaces adjacent to the strand samples to uniformly support the jack bridging assembly. Special care needs to be taken to avoid moving any strand sample after the vibration is complete. [Do *not* re-adjust the height of any strand sample if it is not exactly at the proper height after vibration. A 1/4 to 1/2 in. (6.3 to 13 mm) extra embedment is not significant.]

7. Support racks shall be placed over the test blocks to keep the curing covers from coming in contact with the tops of the strand samples. Curing compound shall be sprayed on the tops of the blocks to prevent shrinkage cracks from occurring in the top surface.

TESTING PROCEDURE

1. The hydraulic jack shall be a pull-jack with a center hole assembly at the end of the ram (similar to those normally used for single-strand stressing). It shall be tested and calibrated to permit loading to 50 kips (222 kN), and shall have a travel of at least 12 in. (305 mm).

2. The bridging device shall be as shown in Fig. E3.

3. On the day after casting the test blocks (with heat curing), the cylinders shall be tested and the concrete strength recorded. Based on results of past testing, the concrete strength can range from 3500 to 5900 psi (24.1 to 40.7 MPa) without affecting the pull-out strength results.

4. The bridge is slipped over each strand to be tested and placed against the concrete surface. The strand chucks are slipped over the strand to the top of the bridge and light pressure is applied to the jack to seat the jaws of the chuck into the strand.

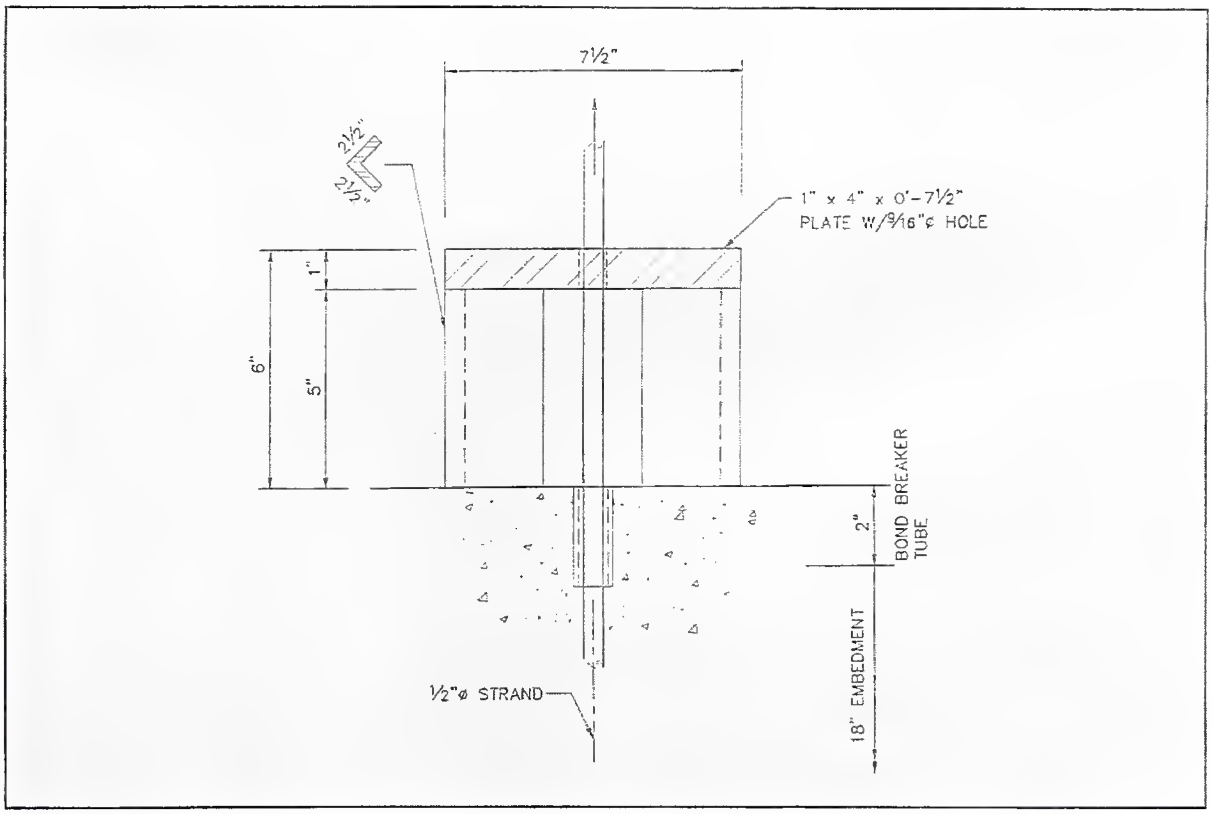


Fig. E3. Bridging device.

5. The jacking load shall be applied in a single increasing application of load at the rate of approximately 20 kips per minute (89 kN per minute) until maximum load is reached and the load gauge indicator can no longer sustain maximum load. Do *not* stop the test at the first sign of movement of the strand sample or for any other reason. The strand samples can pull out as much as 8 to 10 in. (203 to 254 mm) before maximum load is reached with poor bonding strand, and 1 to 2 in. (25.5 to 51 mm) with good bonding strand.

6. The pull-out capacity of the strand sample shall be recorded as the maximum load attained by the strand sample before the load drops off on the gauge and cannot be further increased.

7. The following data shall be recorded for each strand sample:

(a) Maximum capacity (as defined above).

(b) Approximate load at first noticeable movement.

(c) Approximate distance the strand pulls out at maximum load (for general reference, accuracy is not critical).

(d) General description of failure. Typical examples:

(i) Abrupt slip, loud noise. Strand started moving at 35 kips (156 kN). Two wires broke at failure load of 41.2 kips (183 kN).

(ii) Gradual slip, no noise. Strand started moving at approximately 6 kips (26.7 kN).

(iii) Initial movement at approximately 30 kips (133 kN), then abrupt slip at 36.3 kips (161 kN). Loud noise. No broken wires.

(iv) Strand break. All seven wires broke at the chuck.

8. Record data and compute average failure load and standard deviation for each strand group tested. Compare results with minimum requirements for acceptance for pretensioning applications.

APPENDIX B

PROPERTIES OF ENGINEERING MATERIALS USED IN THIS STUDY

PCI Mix Used For Pullout Specimen #1
(4000 psi) NW Cast on 9/16/97

Ingredients	
Cement, lbs	660
Water, lbs	293
Sand, lbs	1100
Course Agg. - Gravel	1900
Coarse Agg. - Stone	
HRWR, ozs/100 lbs	0
NRWR/Ret., ozs/100 lbs	3.9
AEA, ozs/100 lbs	0

Fresh Concrete Properties	
Slump, in.	5
Air Content, %	-
Unit Weight, lbs/cu.ft.	147

Hardened Concrete Properties

Age (days)	Compressive Strength, psi
1	4350
2	4830
4	5150
28	6630

7 ksi NW Mix Used For Pullout Specimen #2
Cast on 9/18/97

Ingredients	
Cement, lbs	840
Water, lbs	266
Sand, lbs	1116
Course Agg. - Gravel	
Coarse Agg. - Stone	1742
HRWR, ozs/100 lbs	19.1
NRWR/Ret., ozs/100 lbs	3.2
AEA, ozs/100 lbs	1.4

Fresh Concrete Properties

Slump, in.	4
Air Content, %	3.7
Unit Weight, lbs/cu.ft.	150

Hardened Concrete Properties

Age (days)	Compressive Strength, psi
1	6990
2	8500
28	11,360

7ksi SLW Mixes Used for the Single Strand Beams & Pullout Specimen
Specimens cast on 10/3/97

Ingredients	Mix 7K-SLW-S	Mix 7K-SLW-P
Cement, lbs	752	818
Water, lbs	251	204
Sand, lbs	1400	1523
Coarse Agg., lbs	649	706
Haydite, lbs	407	440
HRWR, ozs/100 lbs	16	18.5
NRWR/Ret., ozs/100 lbs	2.9	2.7
AEA, ozs/100 lbs	1.33	1.23

Fresh Concrete Properties

Slump, in.	6	3.5
Air Content, %	5	4
Unit Weight, lbs/cu.ft.	135	137

Hardened Concrete Properties

Age, Days	Compressive Strength, psi	
1	5620	7210
3	6320	7700
4	7000	8420
7	7245	8590
14	7800	9730
28	7960	9840

Dynamic Mod. of Elasticity Age, Days	7K, SLW-S (msi)	7K, SLW-P
3	5.09	
7	5.28	
28	5.96	
56	5.68	

Static Mod. of Elasticity Age, Days	7K, SLW-S (a/b) (msi)	7K, SLW-P (msi)
1	3.64/3.76	4.12
3	3.88/3.88	4.37
7	4.01/4.11	4.29
14	4.29/4.29	4.57
28	4.52/4.41	4.89
56	4.74/4.57	4.85

Note: Mix 7k-SLW-S was used for Pullout Specimen #3, as well as single-strand beams 7SLW-IST-3, 7SLW-FWC-3, and the "short" ends of beams 7SLW-IST-2 and 7SLW-FWC-2. Mix 7k-SLW-P was used for single-strand beams 7SLW-IST-1, 7SLW-FWC-1, and the "long" ends of beams 7SLW-IST-2 and 7SLW-FWC-2.

Mix 10k-SLW Used for the Single Strand Beams & Pullout Specimen
Specimens cast on 10/28/97

Ingredients	
Cement, lbs	922
Water, lbs	188.5
Sand, lbs	1383
Coarse Agg., lbs	717
Haydite, lbs	446
HRWR, ozs/100 lbs	26
NRWR/Ret., ozs/100 lbs	2.78
AEA, ozs/100 lbs	1.12

Fresh Concrete Properties	
Slump, in.	5
Air Content, %	5
Unit Weight, lbs/cu.ft.	138

Hardened Concrete Properties

	Beams	*Pullout Specimen*
Age, Days	Compressive Strength, psi	
1	-	-
3	8950	-
7	9990	11,920
14	10,600	-
28	12,060	13,210
56	-	14,260
91	12,760	14,070

Dynamic Mod. of Elasticity Age, Days	10K, SLW (msi)
3	5.84
7	6.06
28	6.27

Static Mod. of Elasticity Age, Days	10K, SLW (msi)
1	-
3	4.74
7	4.77
14	5.04
28	5.23

T-Beams IST-1, IST-2, FWC
Cast on 5/22/98 using Mix 7k-SLW

Concrete Compressive Strengths (psi)

Age, days	*Mix1*	*Mix2*
1	4180	3960
3	6870	6380
7	8320	7470
14	8460	8210
26	8870	8380
28	9320	8420

Note: T-Beams IST1 and FWC contained Mix 1
T-Beam IST2 contained Mix2

T-Beams FWC-3", FWC-6", FWC-15"
Cast on 6/30/98 using Mix 7k-SLW

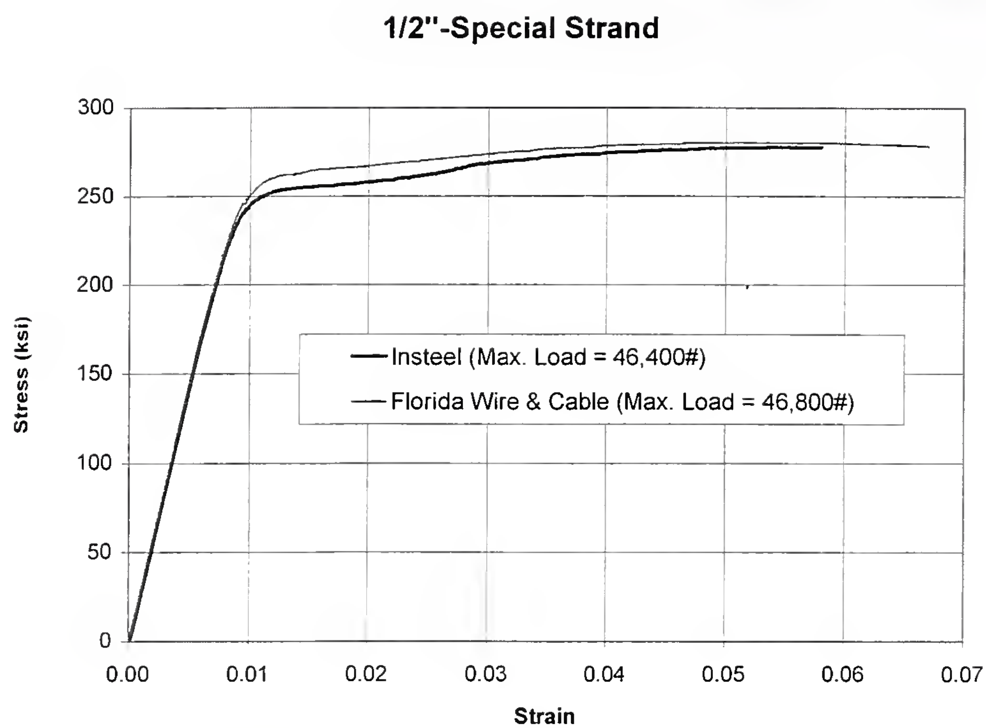
Concrete Compressive Strengths (psi)

Age, days	*Mix1*	*Mix2*
1	5040	3710
3	7080	5650
7	7560	5760
14	7900	6530
21	8260	6660
28	8730	7100

Note: T-Beams FWC-3" and FWC-6" contained Mix1
T-Beam FWC-15" contained Mix2

T-Beams 10SLW-0.6"
Cast on 7/14/98 using Mix 10k-SLW

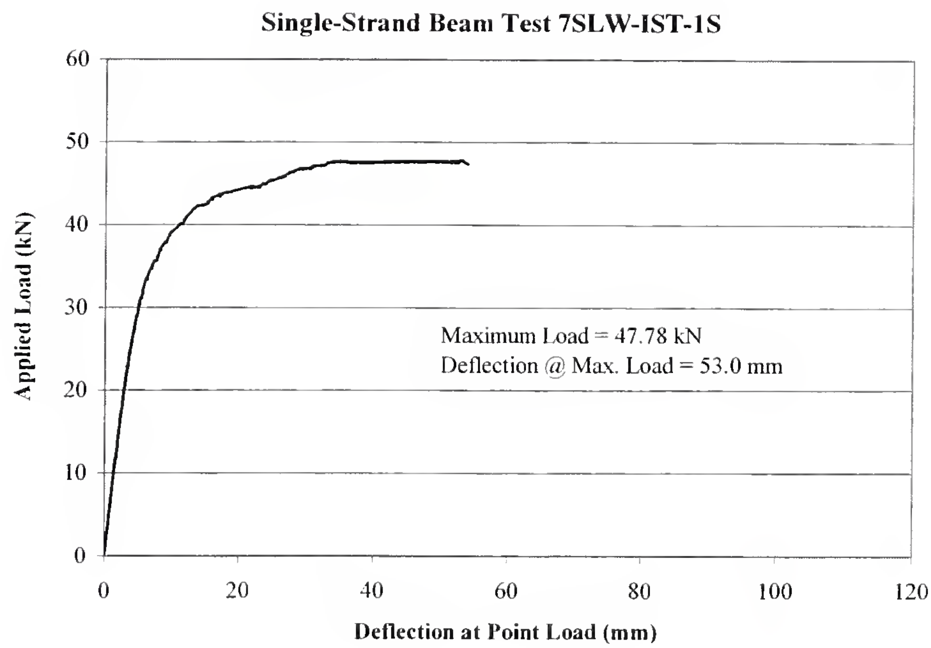
Age, days	Compressive Strength (psi)
1	7110
3	8290
7	9340
14	9950

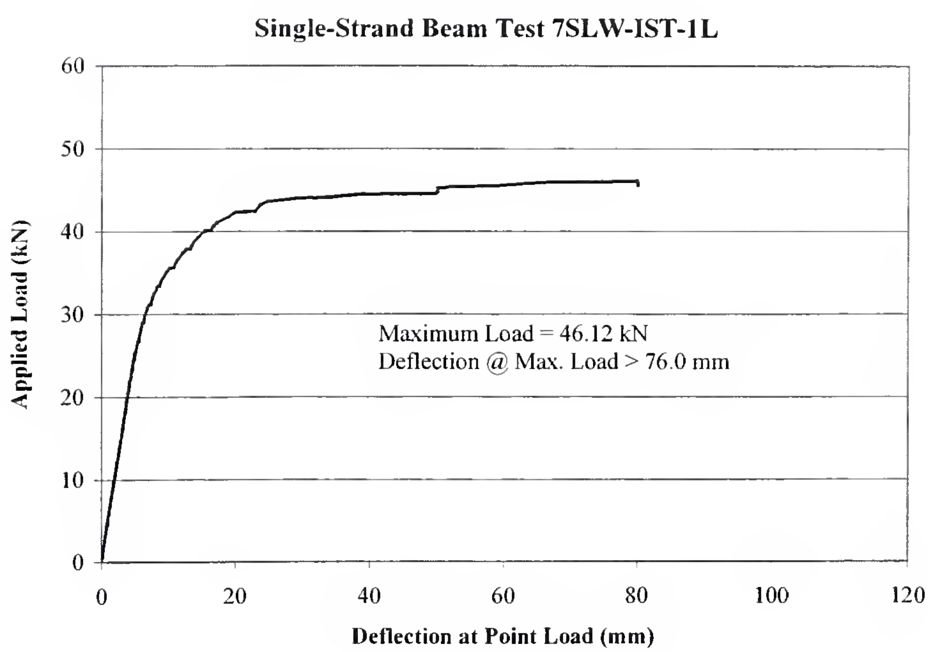


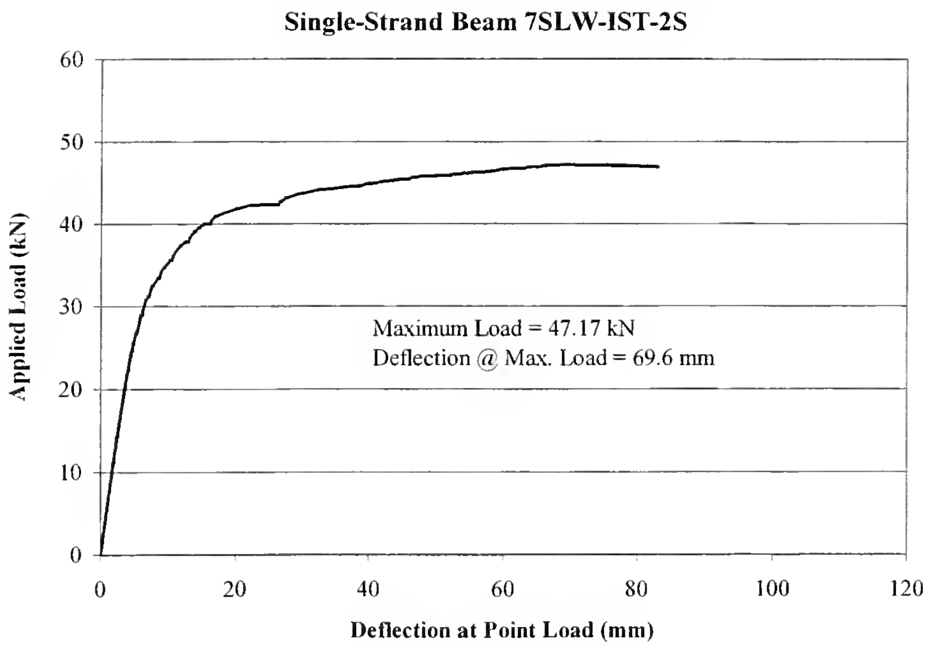
Measures stress-strain response for the 1/2”-Special (13.3 mm) strand Used in this study.

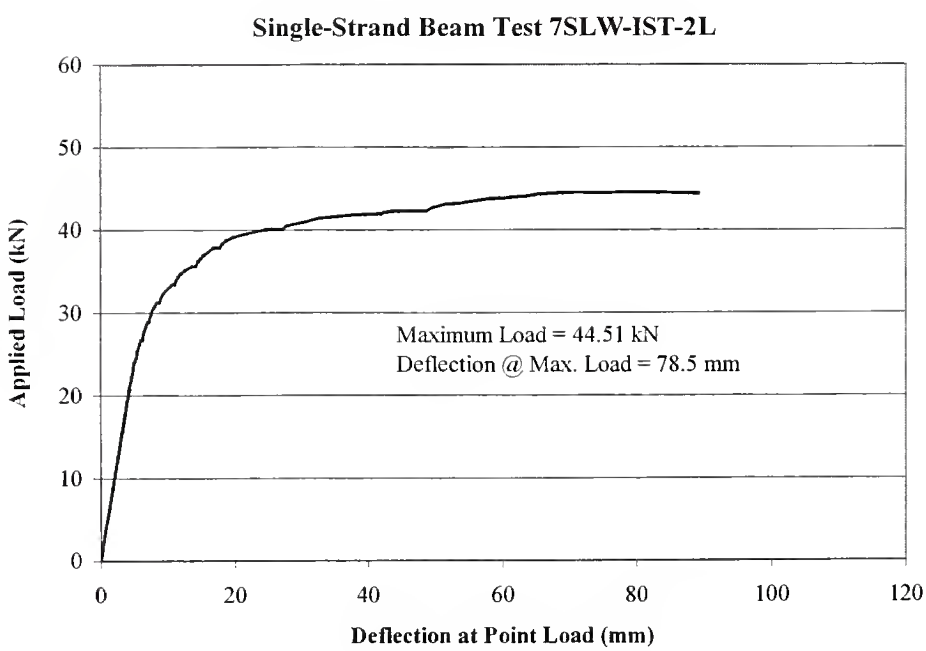
APPENDIX C

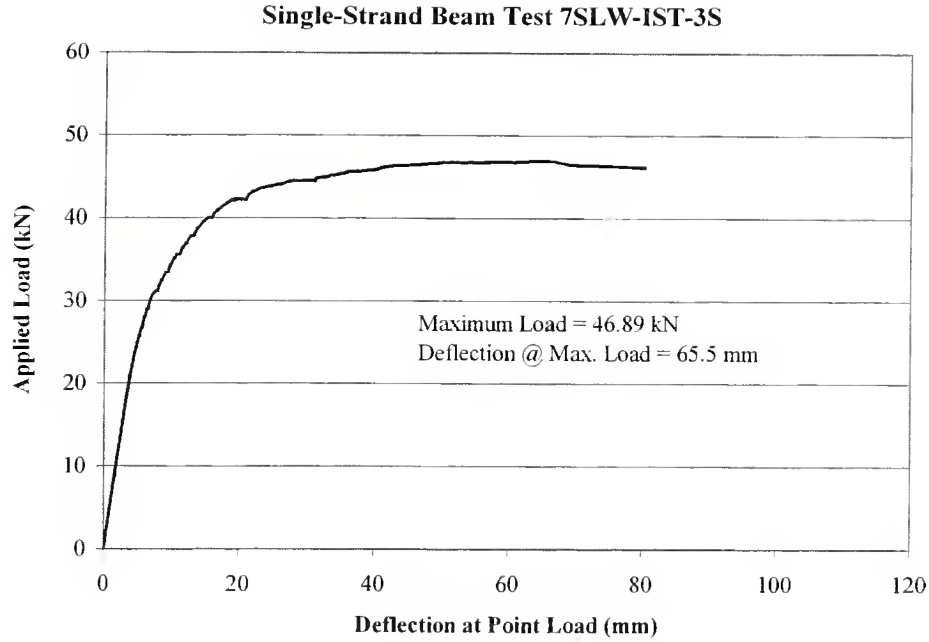
LOAD-DEFLECTION PLOTS FOR ALL DEVELOPMENT-LENGTH SPECIMENS

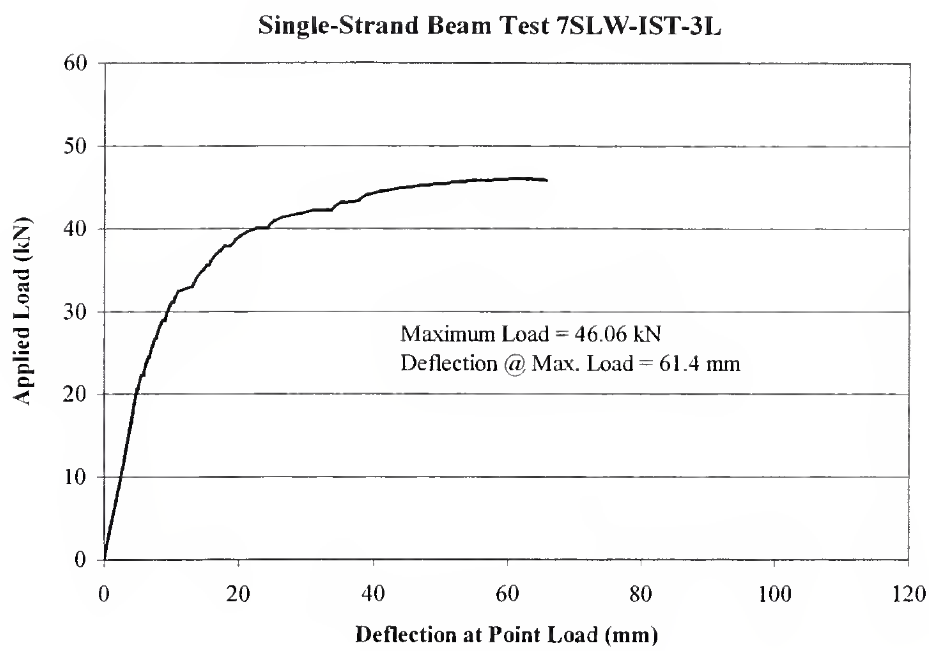

Single-Strand Beam Test 7SLW-IST-1S
Applied Load (kN)
60
50
40
30
20
10
0
0
20
40
60
80
100
120
Deflection at Point Load (mm)
Maximum Load = 47.78 kN
Deflection @ Max. Load = 53.0 mm

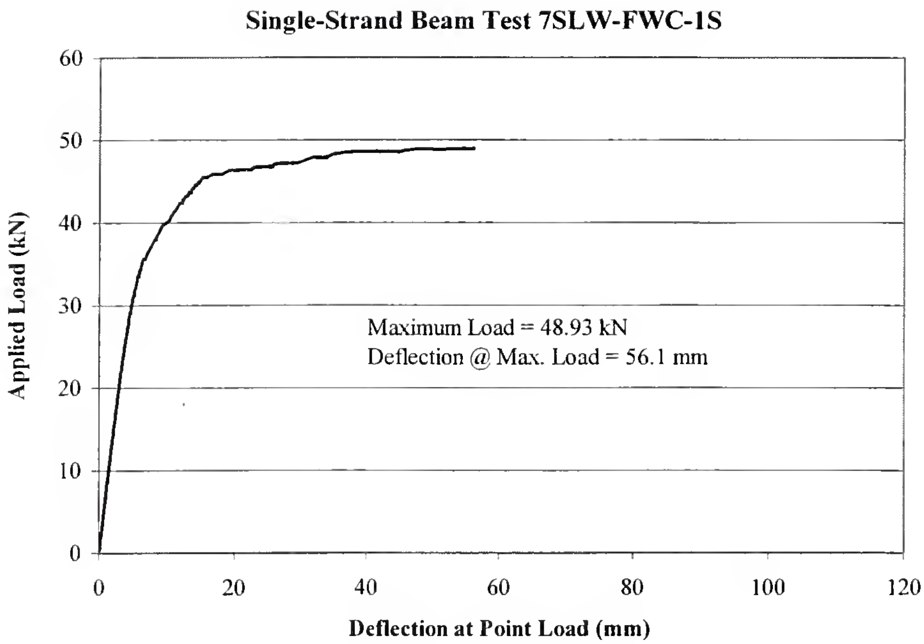

Single-Strand Beam Test 7SLW-IST-1L
Applied Load (kN)
60
50
40
30
20
10
0
0
20
40
60
80
100
120
Deflection at Point Load (mm)
Maximum Load = 46.12 kN
Deflection @ Max. Load > 76.0 mm

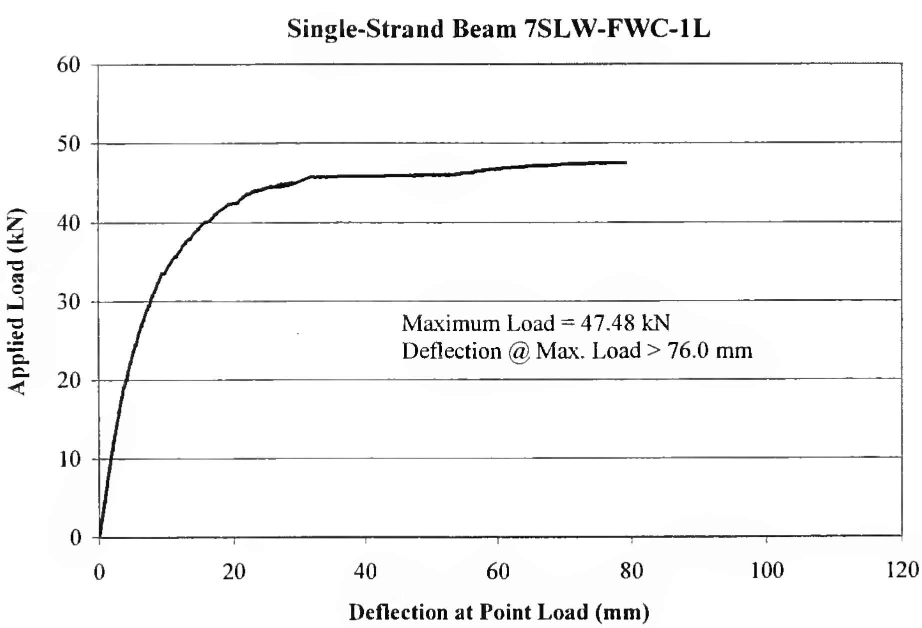

Single-Strand Beam 7SLW-IST-2S
Applied Load (kN)
Deflection at Point Load (mm)
Maximum Load = 47.17 kN
Deflection @ Max. Load = 69.6 mm
0
10
20
30
40
50
60
0
20
40
60
80
100
120

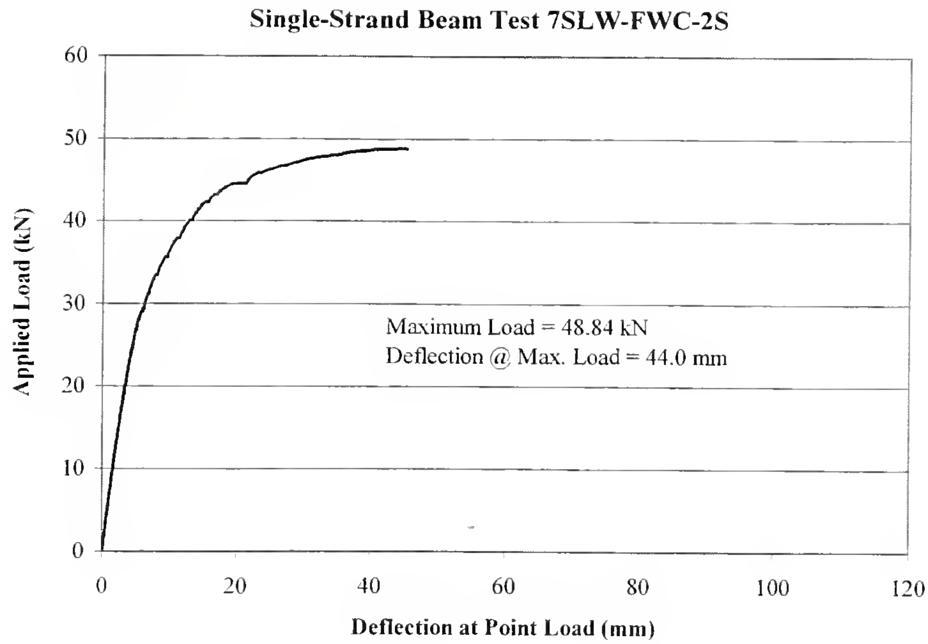

Single-Strand Beam Test 7SLW-IST-2L
Applied Load (kN)
Deflection at Point Load (mm)
Maximum Load = 44.51 kN
Deflection @ Max. Load = 78.5 mm
0
10
20
30
40
50
60
0
20
40
60
80
100
120

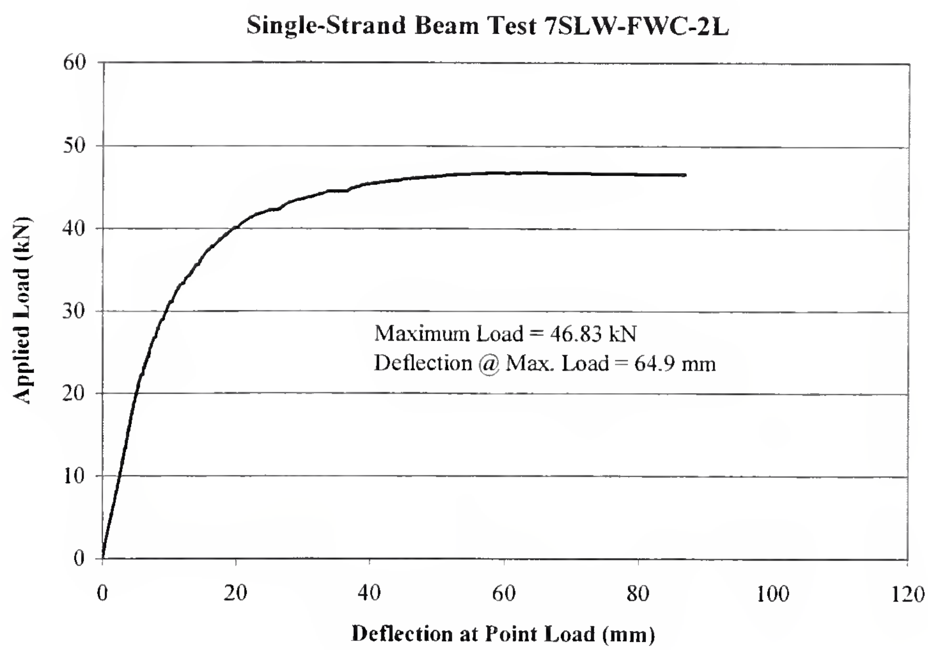

Single-Strand Beam Test 7SLW-IST-3S
Applied Load (kN)
60
50
40
30
20
10
0
0
20
40
60
80
100
120
Deflection at Point Load (mm)
Maximum Load = 46.89 kN
Deflection @ Max. Load = 65.5 mm

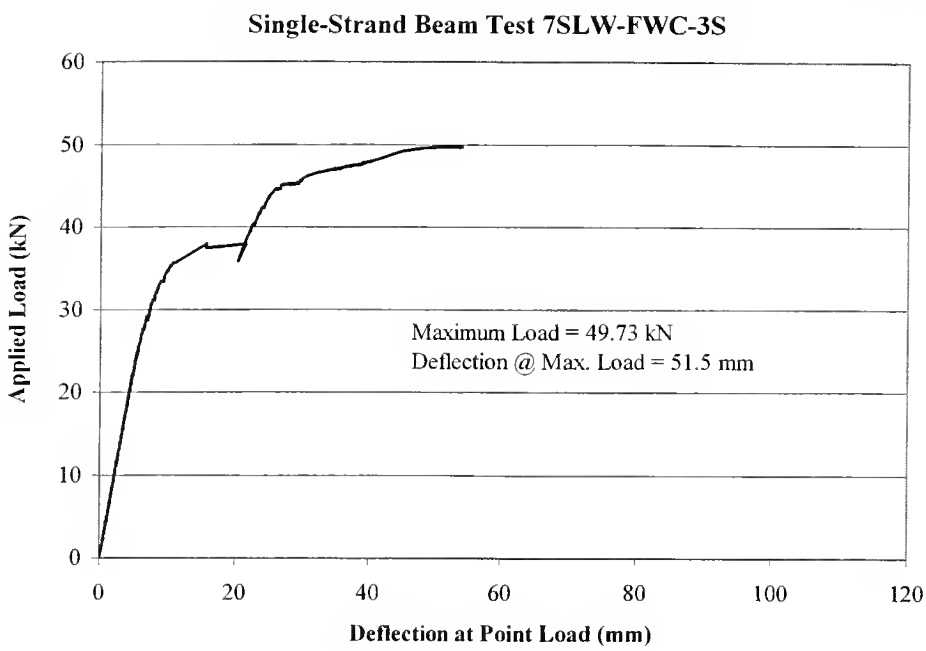

Single-Strand Beam Test 7SLW-IST-3L
Applied Load (kN)
60
50
40
30
20
10
0
0
20
40
60
80
100
120
Deflection at Point Load (mm)
Maximum Load = 46.06 kN
Deflection @ Max. Load = 61.4 mm

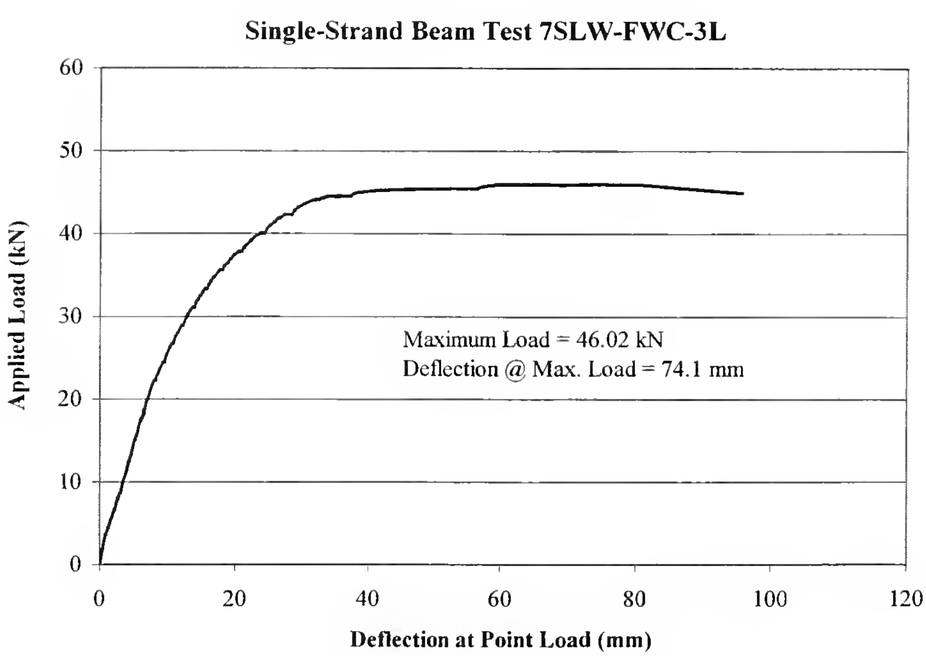

Single-Strand Beam Test 7SLW-FWC-1S
Applied Load (kN)
60
50
40
30
20
10
0
Maximum Load = 48.93 kN
Deflection @ Max. Load = 56.1 mm
0
20
40
60
80
100
120
Deflection at Point Load (mm)

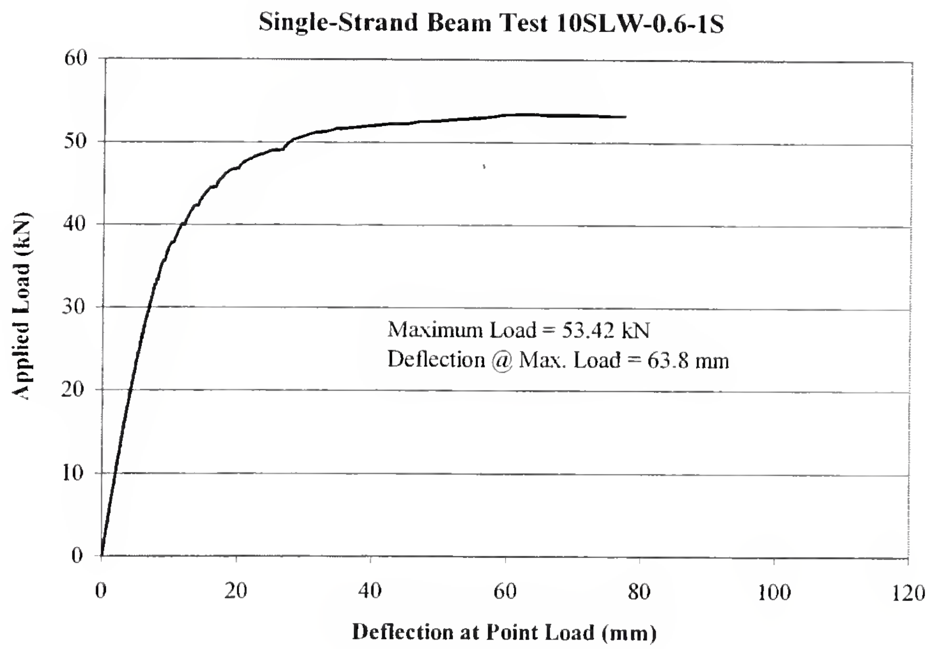

Single-Strand Beam 7SLW-FWC-1L
Applied Load (kN)
60
50
40
30
20
10
0
Maximum Load = 47.48 kN
Deflection @ Max. Load > 76.0 mm
0
20
40
60
80
100
120
Deflection at Point Load (mm)

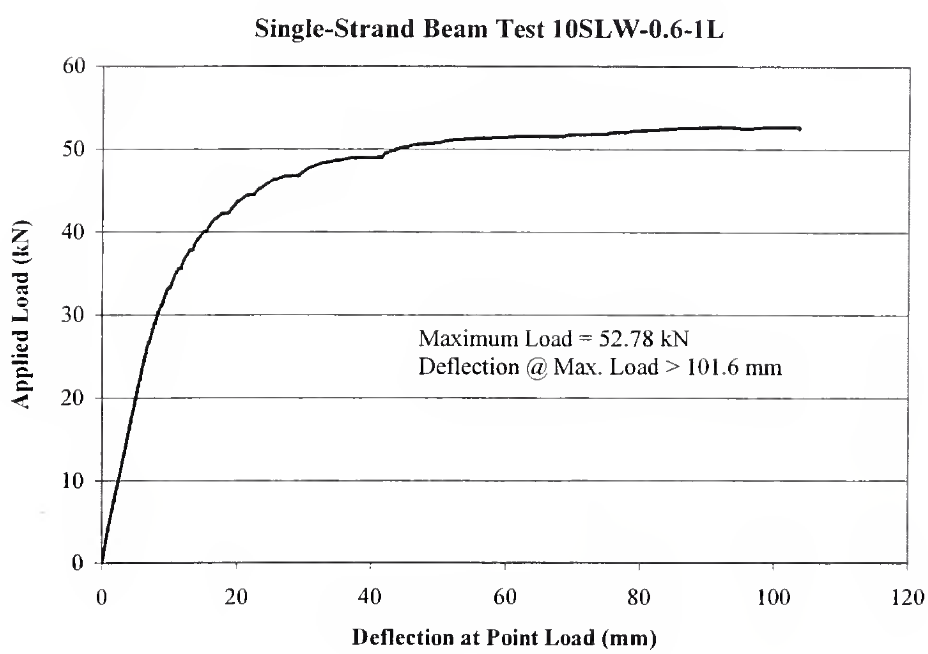

Single-Strand Beam Test 7SLW-FWC-2S
Applied Load (kN)
60
50
40
30
20
10
0
0
20
40
60
80
100
120
Deflection at Point Load (mm)
Maximum Load = 48.84 kN
Deflection @ Max. Load = 44.0 mm

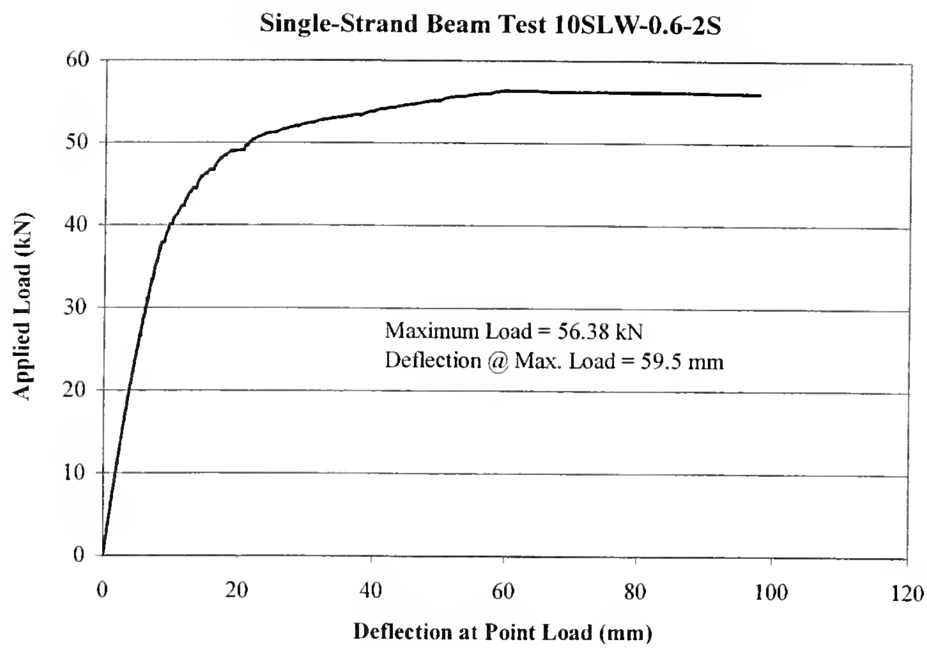

Single-Strand Beam Test 7SLW-FWC-2L
Applied Load (kN)
60
50
40
30
20
10
0
0
20
40
60
80
100
120
Deflection at Point Load (mm)
Maximum Load = 46.83 kN
Deflection @ Max. Load = 64.9 mm

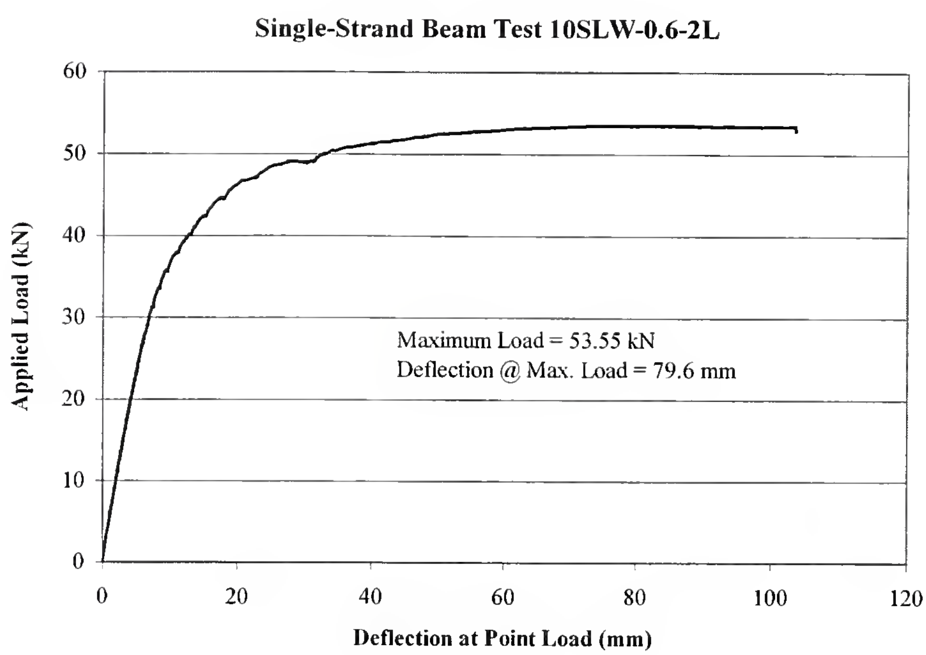

Single-Strand Beam Test 7SLW-FWC-3S
Applied Load (kN)
Deflection at Point Load (mm)
Maximum Load = 49.73 kN
Deflection @ Max. Load = 51.5 mm
0
10
20
30
40
50
60
0
20
40
60
80
100
120

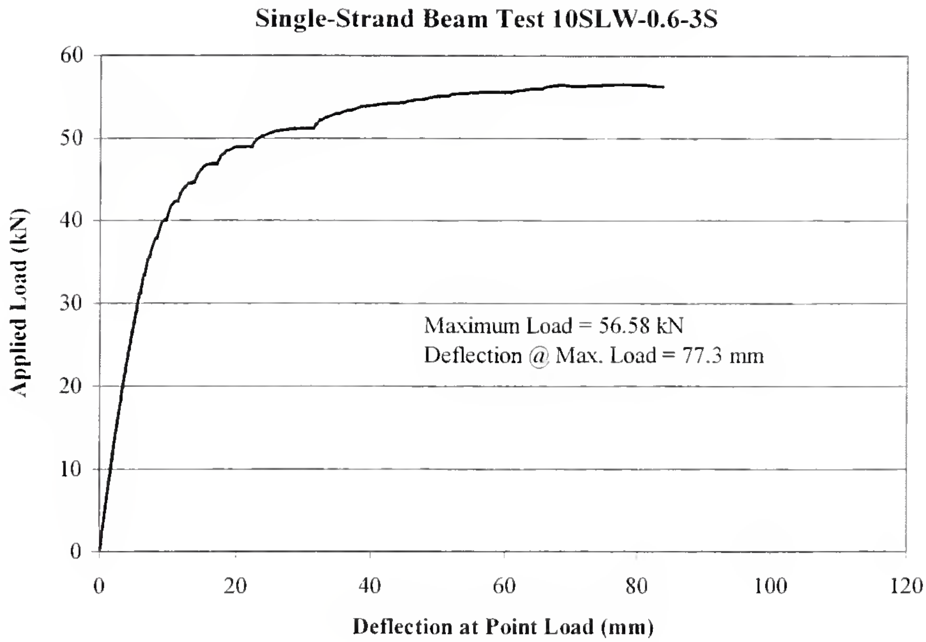

Single-Strand Beam Test 7SLW-FWC-3L
Applied Load (kN)
Deflection at Point Load (mm)
Maximum Load = 46.02 kN
Deflection @ Max. Load = 74.1 mm
0
10
20
30
40
50
60
0
20
40
60
80
100
120

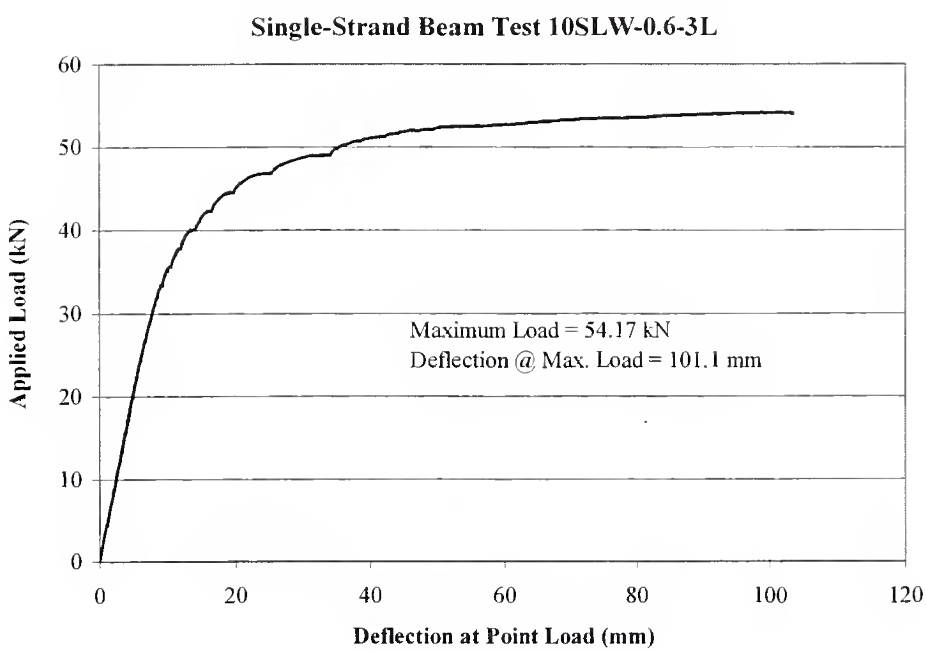

Single-Strand Beam Test 10SLW-0.6-1S
Applied Load (kN)
0
10
20
30
40
50
60
Maximum Load = 53.42 kN
Deflection @ Max. Load = 63.8 mm
0
20
40
60
80
100
120
Deflection at Point Load (mm)

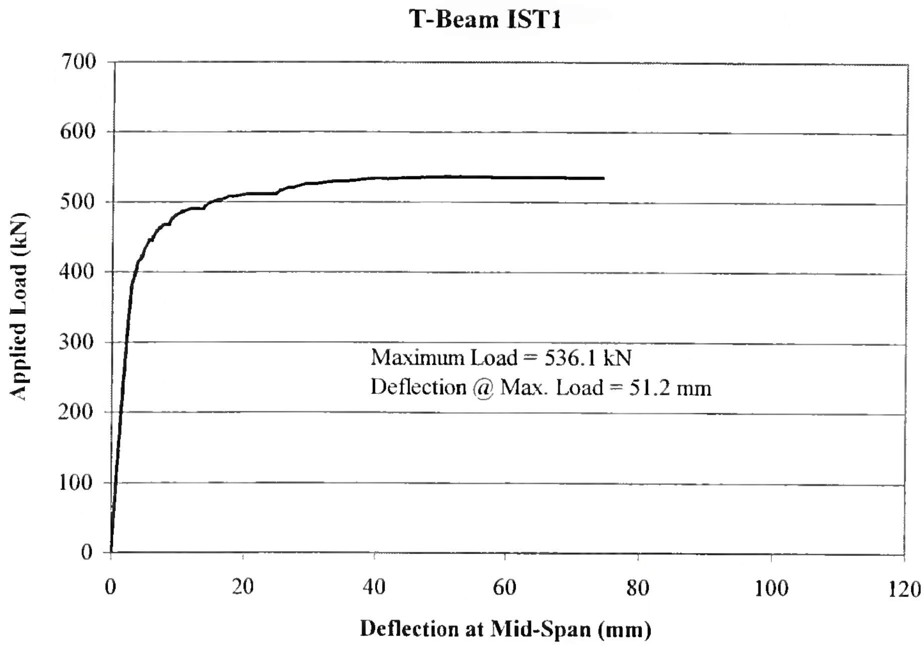

Single-Strand Beam Test 10SLW-0.6-1L
Applied Load (kN)
0
10
20
30
40
50
60
Maximum Load = 52.78 kN
Deflection @ Max. Load > 101.6 mm
0
20
40
60
80
100
120
Deflection at Point Load (mm)

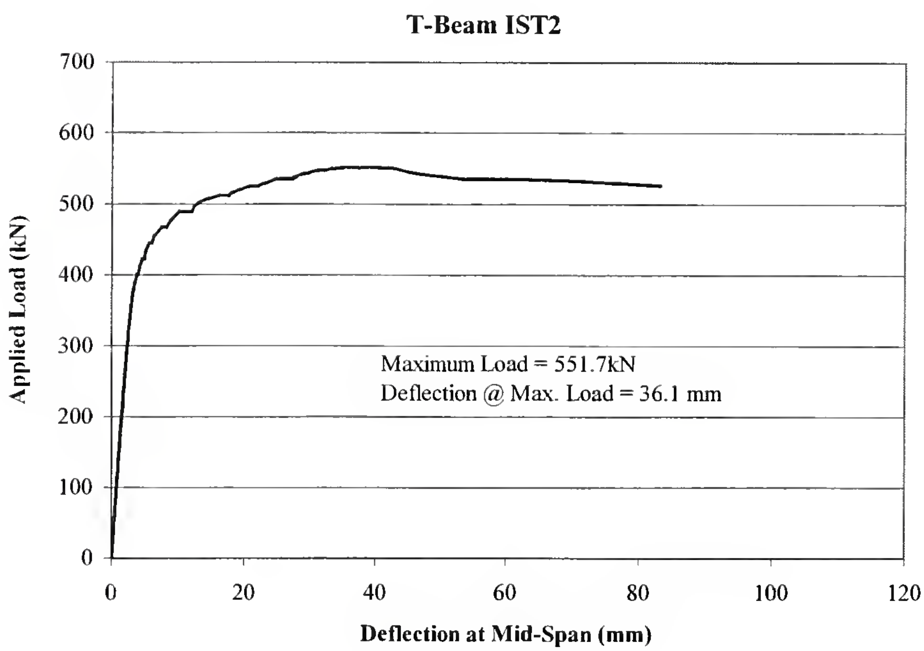

Single-Strand Beam Test 10SLW-0.6-2S
Applied Load (kN)
Deflection at Point Load (mm)
Maximum Load = 56.38 kN
Deflection @ Max. Load = 59.5 mm
0
10
20
30
40
50
60
0
20
40
60
80
100
120


Single-Strand Beam Test 10SLW-0.6-2L
Applied Load (kN)
Deflection at Point Load (mm)
Maximum Load = 53.55 kN
Deflection @ Max. Load = 79.6 mm
0
10
20
30
40
50
60
0
20
40
60
80
100
120

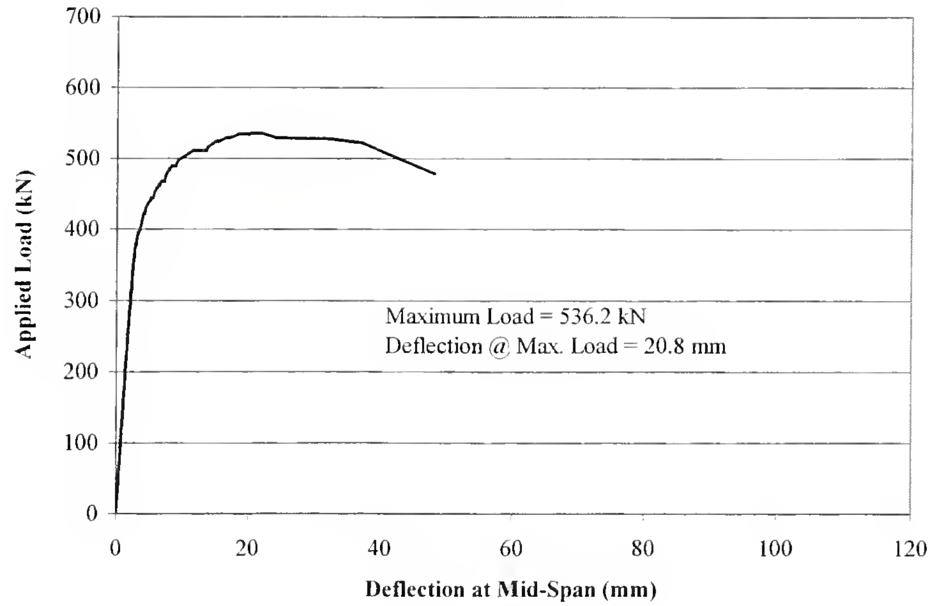

Single-Strand Beam Test 10SLW-0.6-3S
Applied Load (kN)
Deflection at Point Load (mm)
Maximum Load = 56.58 kN
Deflection @ Max. Load = 77.3 mm
0
10
20
30
40
50
60
0
20
40
60
80
100
120


Single-Strand Beam Test 10SLW-0.6-3L
Applied Load (kN)
Deflection at Point Load (mm)
Maximum Load = 54.17 kN
Deflection @ Max. Load = 101.1 mm
0
10
20
30
40
50
60
0
20
40
60
80
100
120

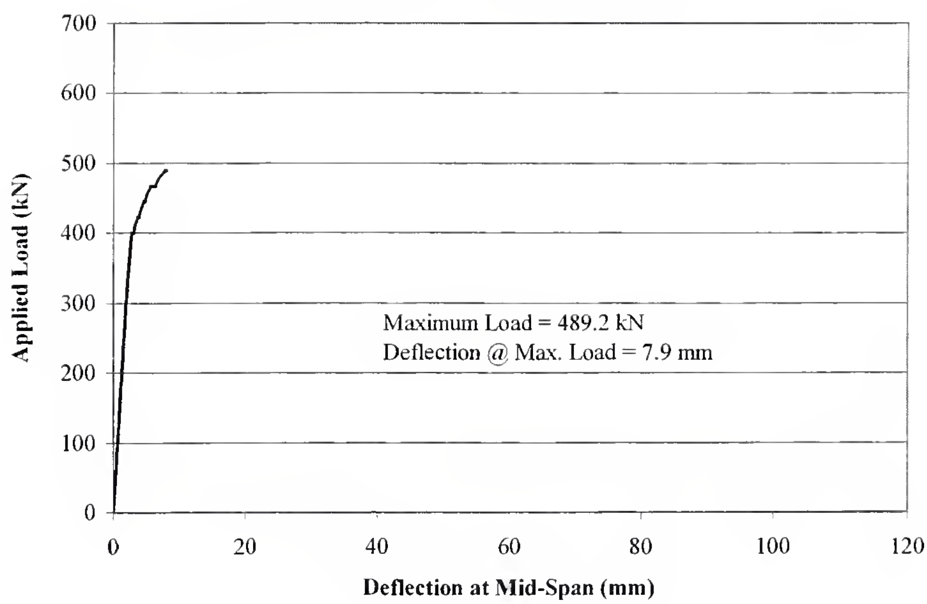

T-Beam IST1
700
600
500
400
300
200
100
0
Applied Load (kN)
Maximum Load = 536.1 kN
Deflection @ Max. Load = 51.2 mm
0
20
40
60
80
100
120
Deflection at Mid-Span (mm)

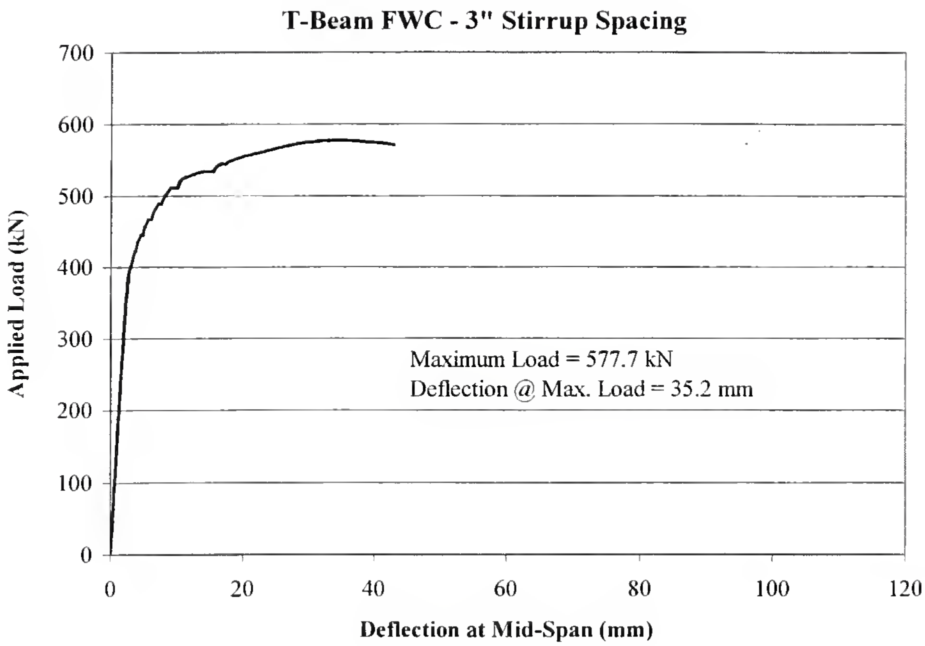

T-Beam IST2
700
600
500
400
300
200
100
0
Applied Load (kN)
Maximum Load = 551.7kN
Deflection @ Max. Load = 36.1 mm
0
20
40
60
80
100
120
Deflection at Mid-Span (mm)

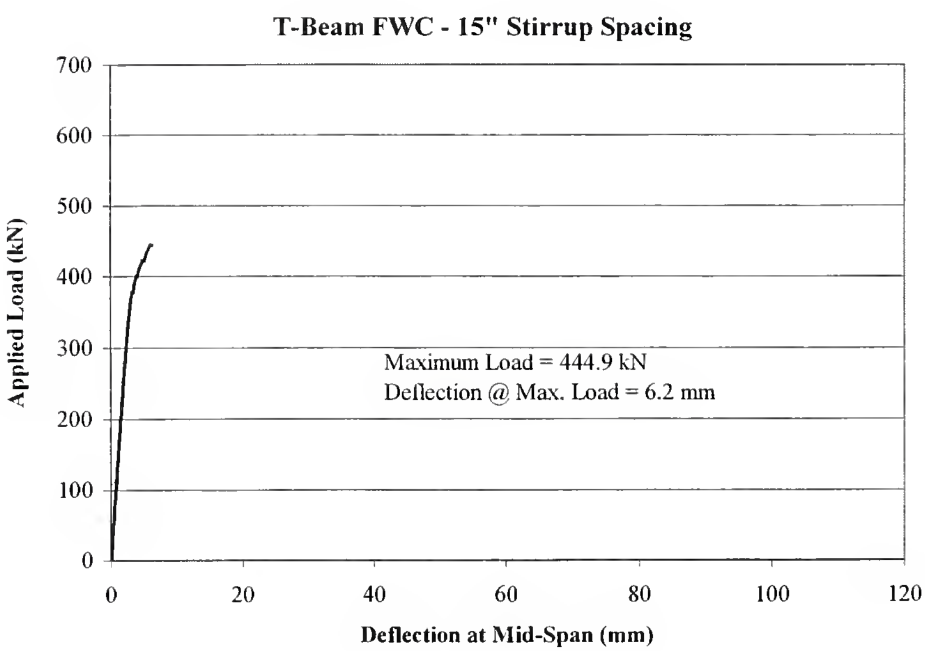
T-Beam FWC

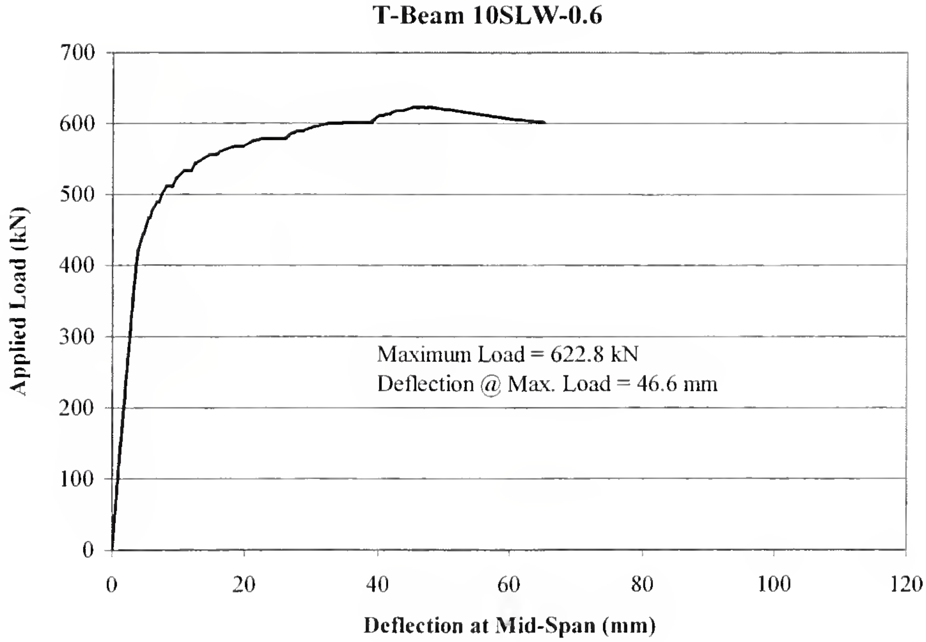


T-Beam FWC - 6" Stirrup Spacing



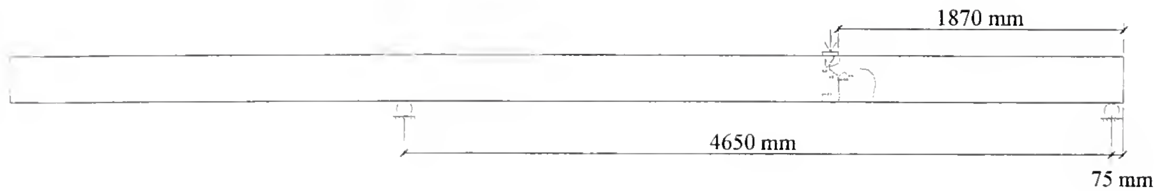

T-Beam FWC - 3" Stirrup Spacing
Applied Load (kN)
700
600
500
400
300
200
100
0
0
20
40
60
80
100
120
Deflection at Mid-Span (mm)
Maximum Load = 577.7 kN
Deflection @ Max. Load = 35.2 mm

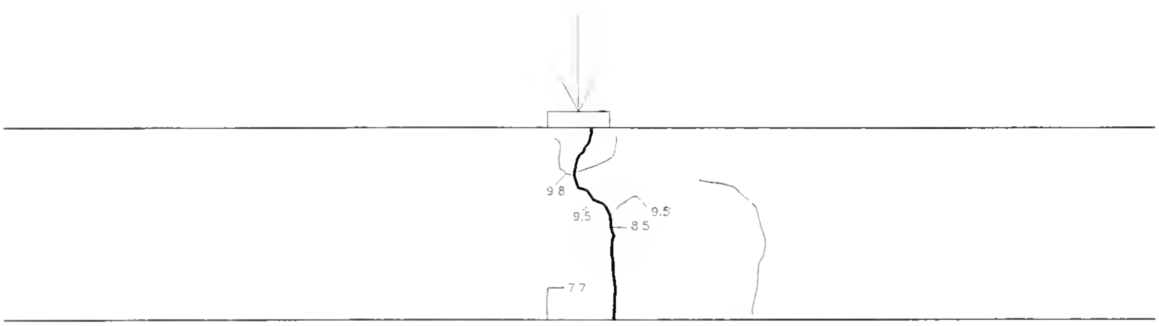

T-Beam FWC - 15" Stirrup Spacing
Applied Load (kN)
700
600
500
400
300
200
100
0
0
20
40
60
80
100
120
Deflection at Mid-Span (mm)
Maximum Load = 444.9 kN
Deflection @ Max. Load = 6.2 mm


T-Beam 10SLW-0.6
Applied Load (kN)
700
600
500
400
300
200
100
0
Maximum Load = 622.8 kN
Deflection @ Max. Load = 46.6 mm
0
20
40
60
80
100
120
Deflection at Mid-Span (mm)

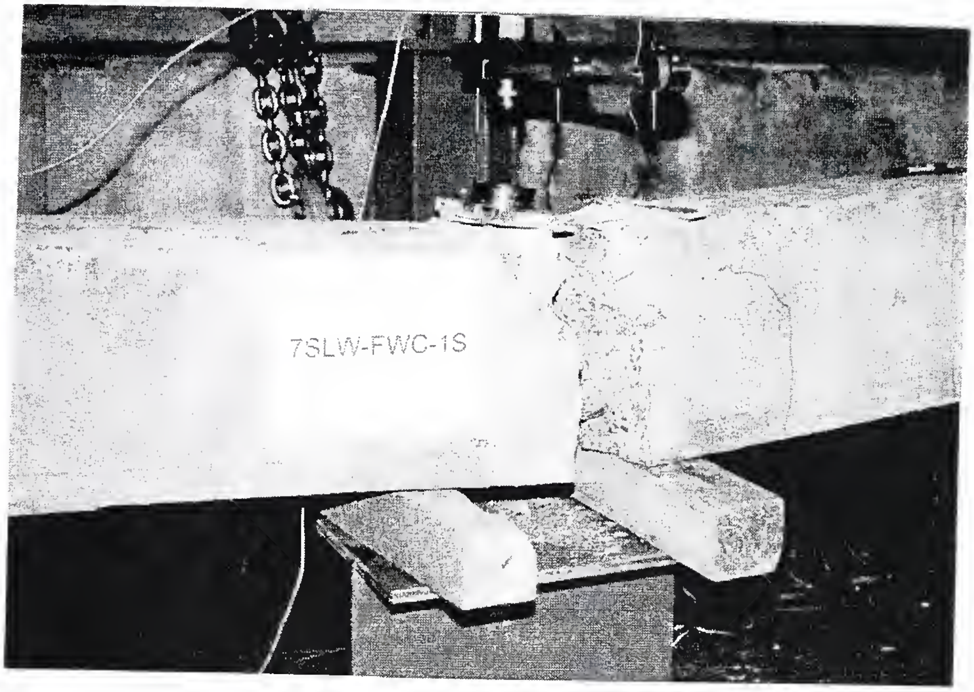
APPENDIX D

SINGLE-STRAND BEAM TESTS -- CRACK PATTERNS AND PHOTOS



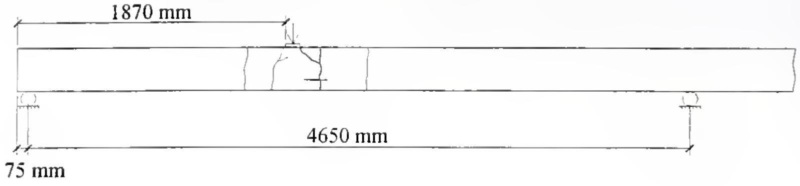


7SLW-FWC-1S – Cracks at Failure

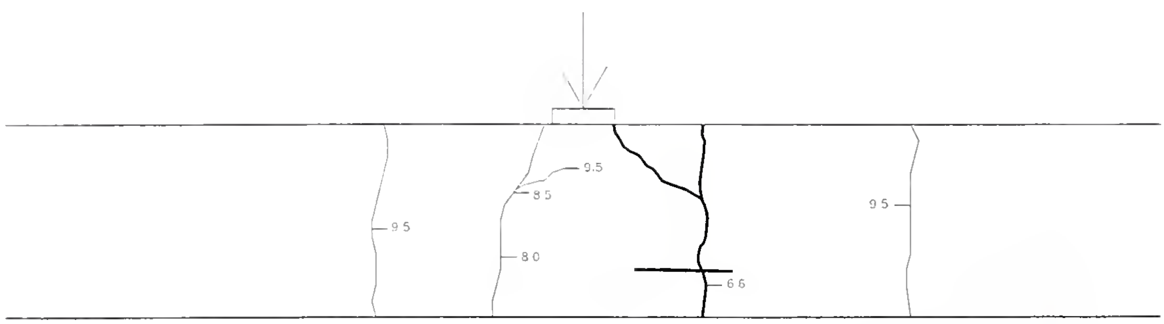


7SLW-FWC-1S

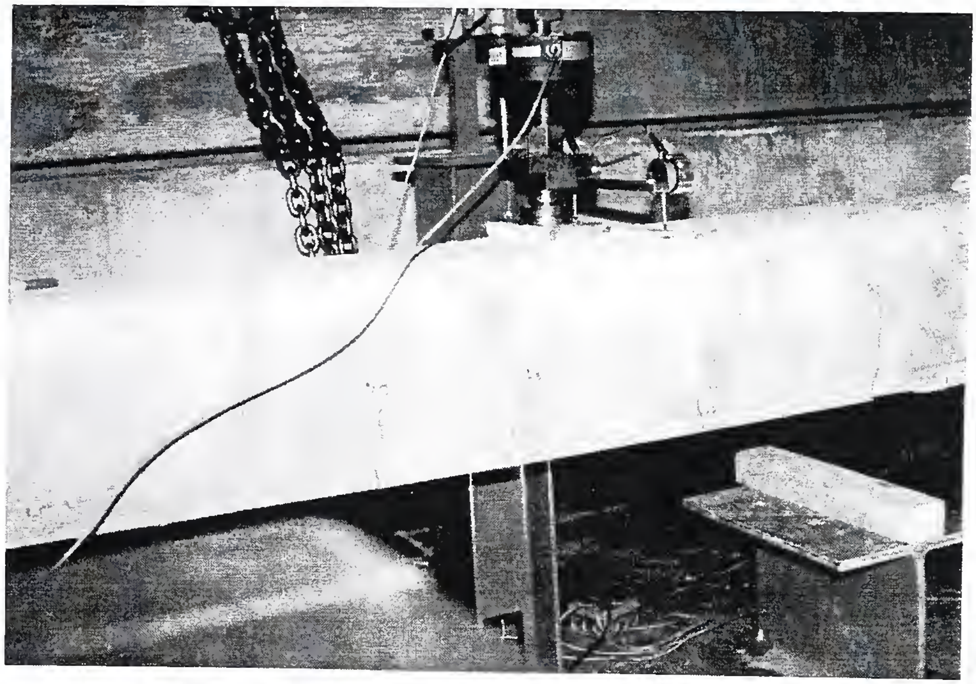
7SLW-FWC-1S



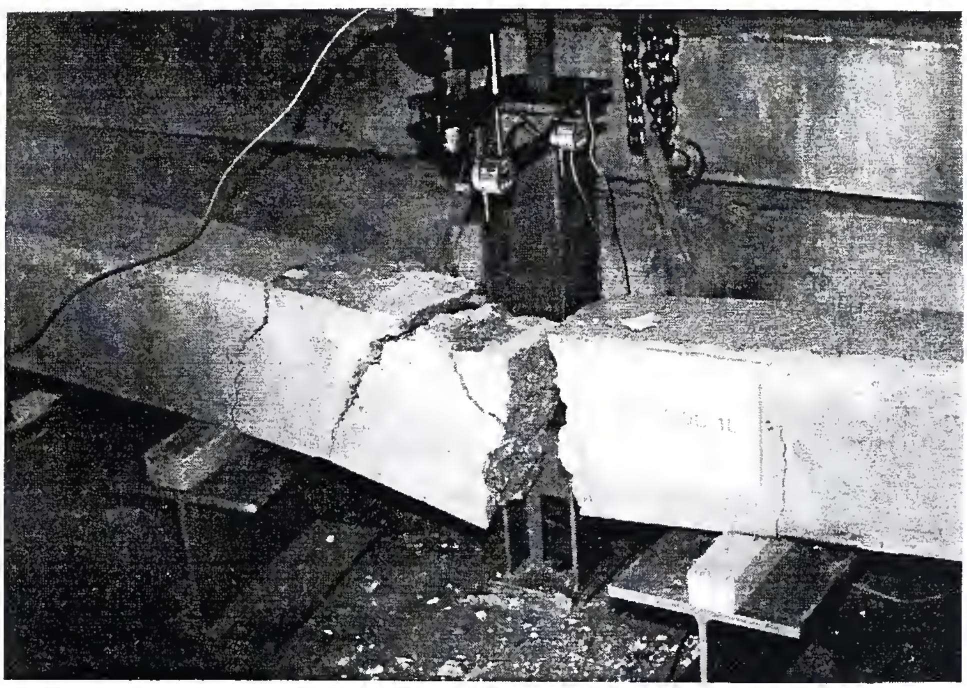
7SLW-FWC-1L



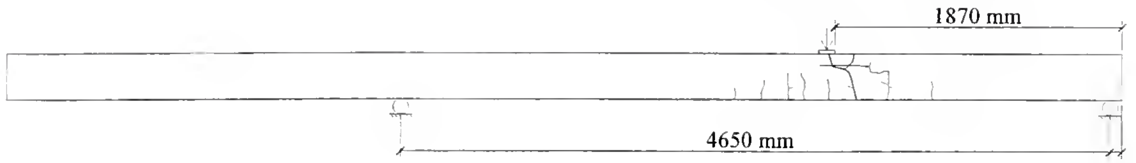
7SLW-FWC-1L – Cracks at Failure



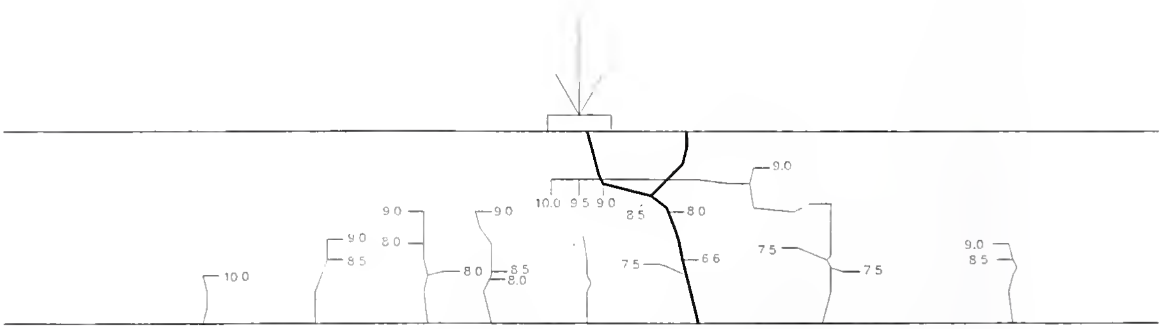
7SLW-FWC-1L



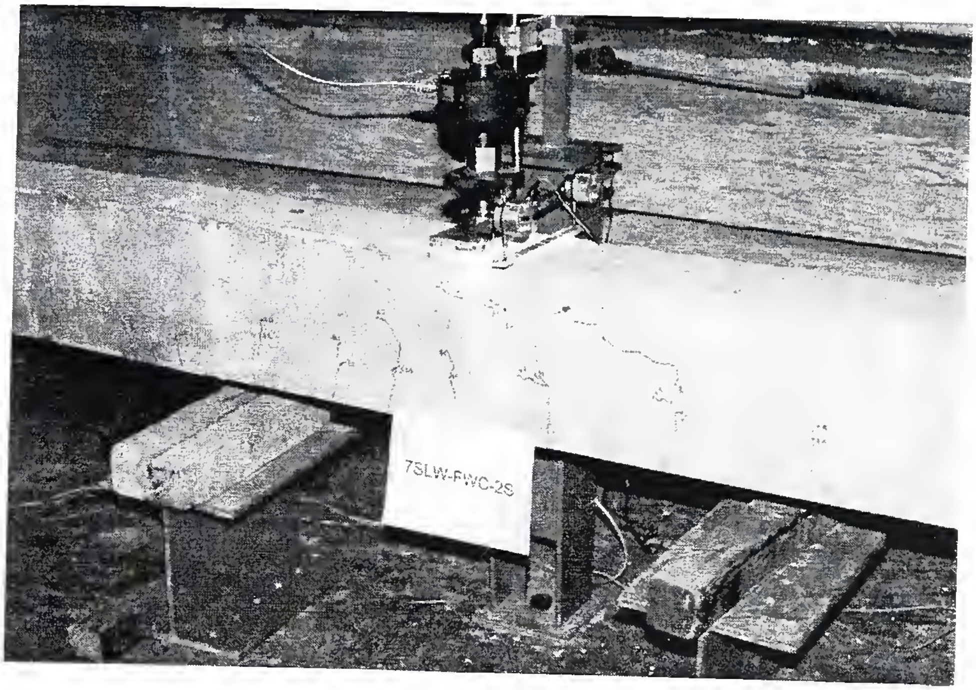
7SLW-FWC-1L



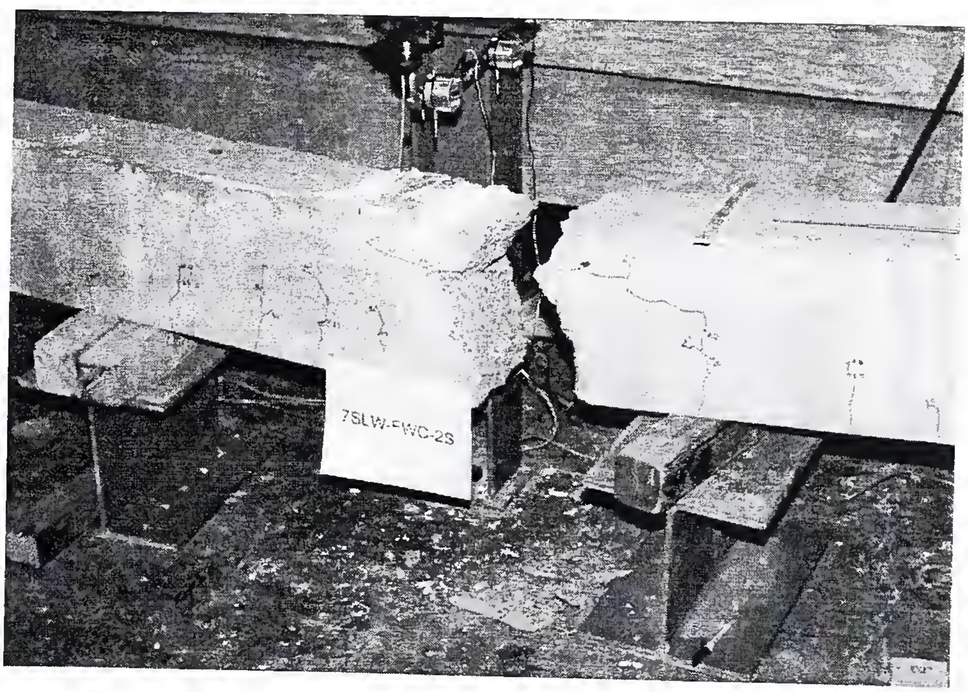
7SLW-FWC-2S



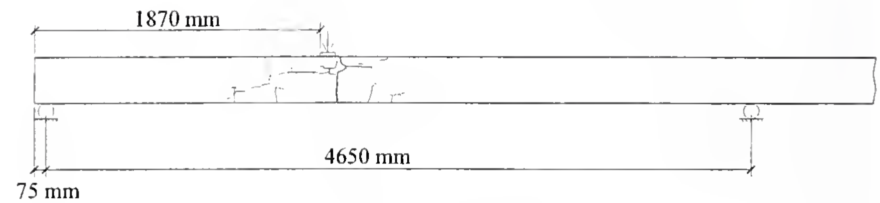
7SLW-FWC-2S – Cracks at Failure



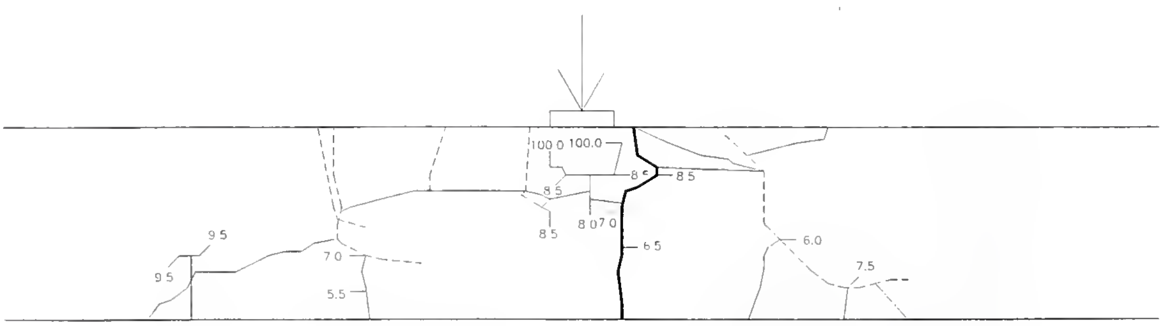
7SLW-FWC-2S



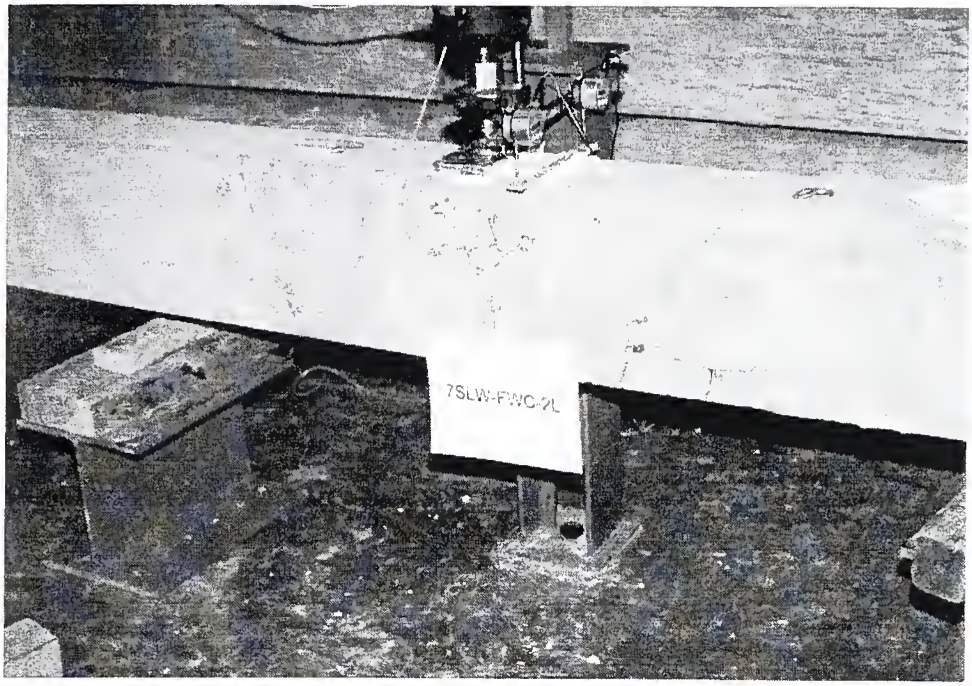
7SLW-FWC-2S



7SLW-FWC-2L

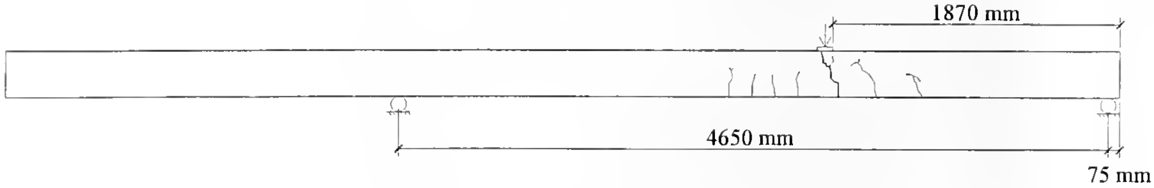


7SLW-FWC-2L – Cracks at Failure

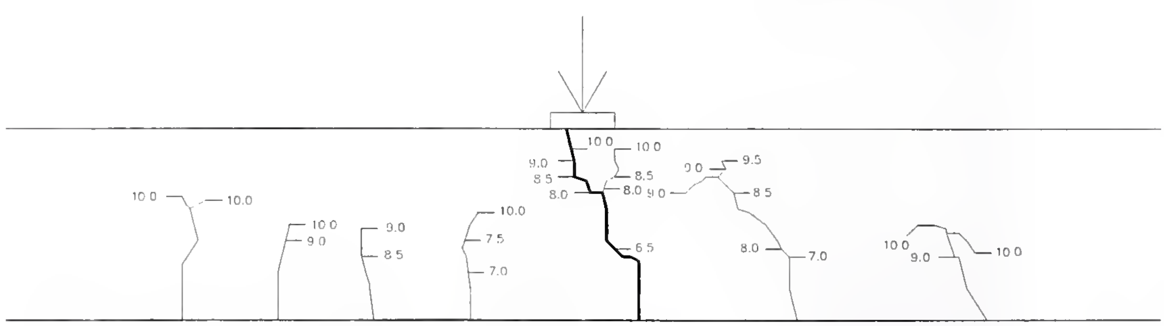


7SLW-FWC-2L

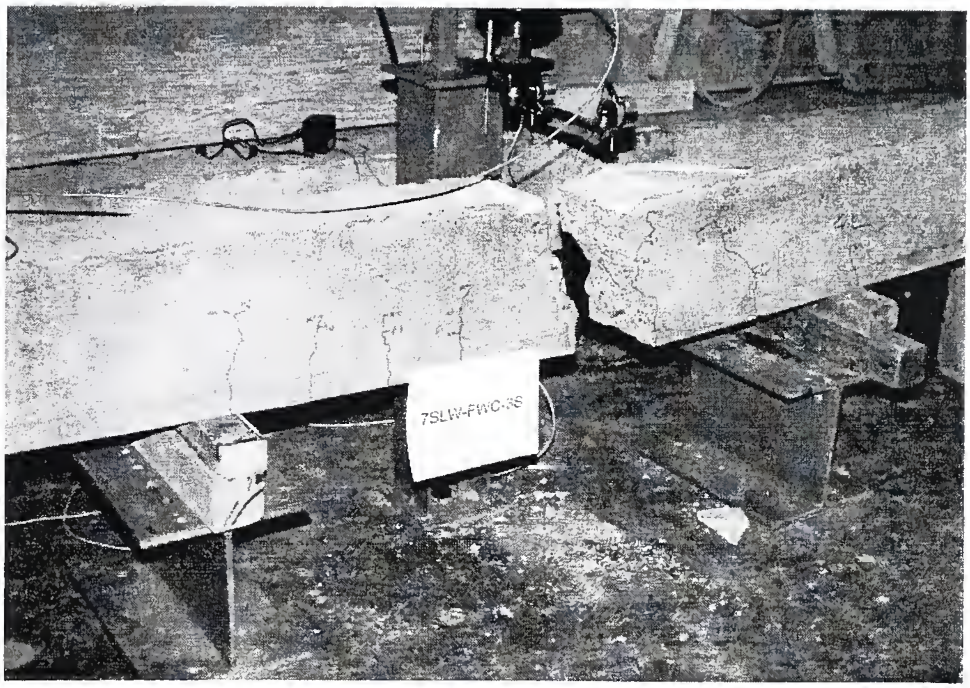
7SLW-FWC-2L



7SLW-FWC-3S

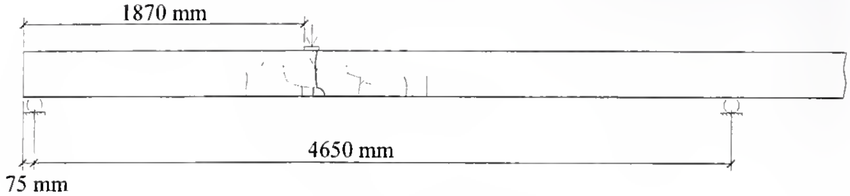


7SLW-FWC-3S – Cracks at Failure

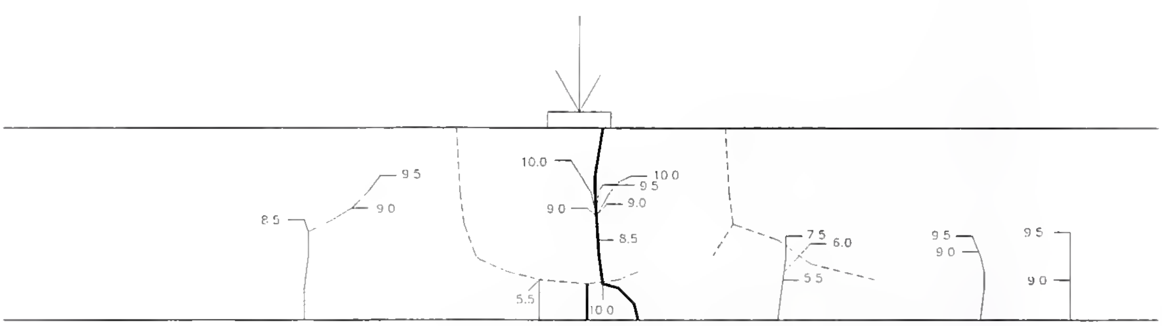


7SLW-FWC-3S

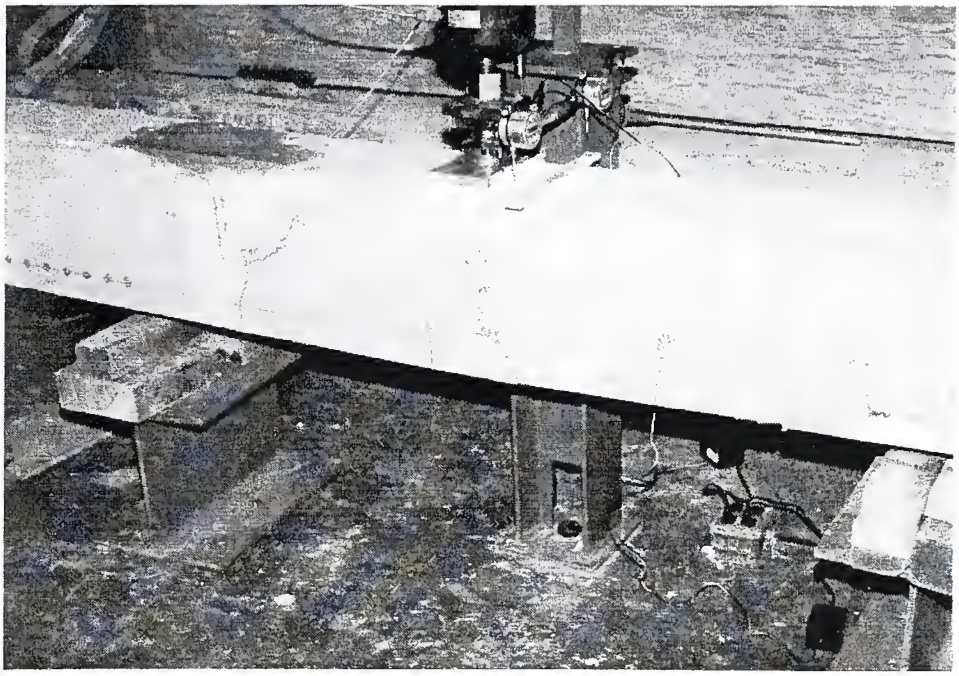
7SLW-FWC-3S – Failure by Strand Rupture



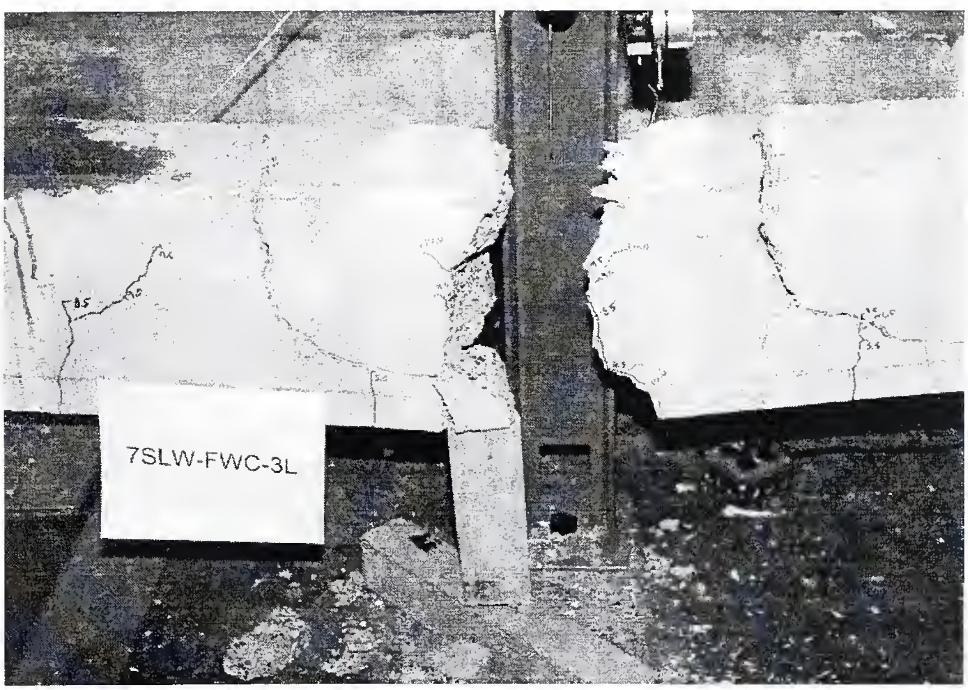
7SLW-FWC-3L



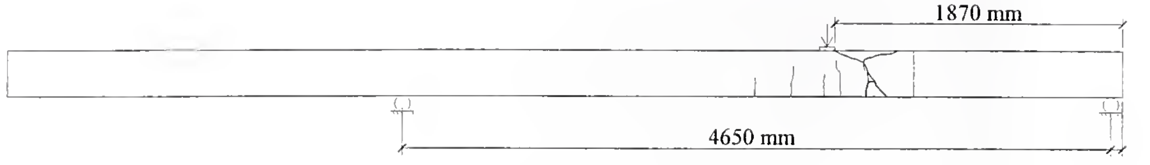
7SLW-FWC-3L – Cracks at Failure



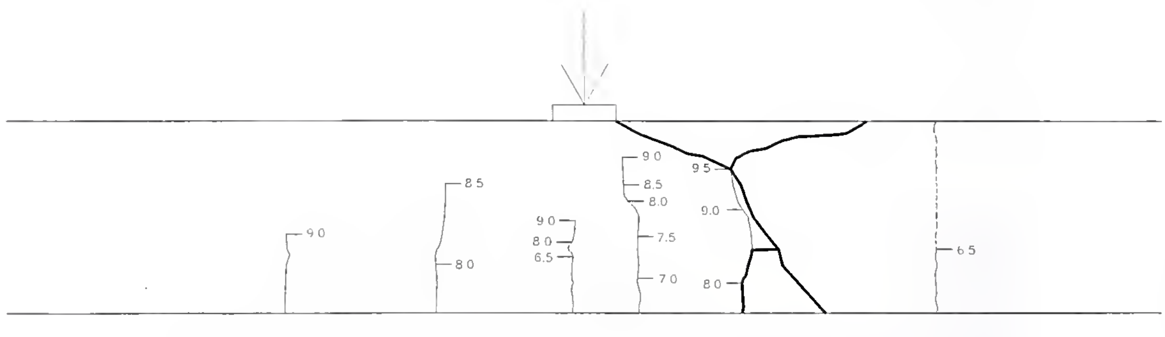
7SLW-FWC-3L



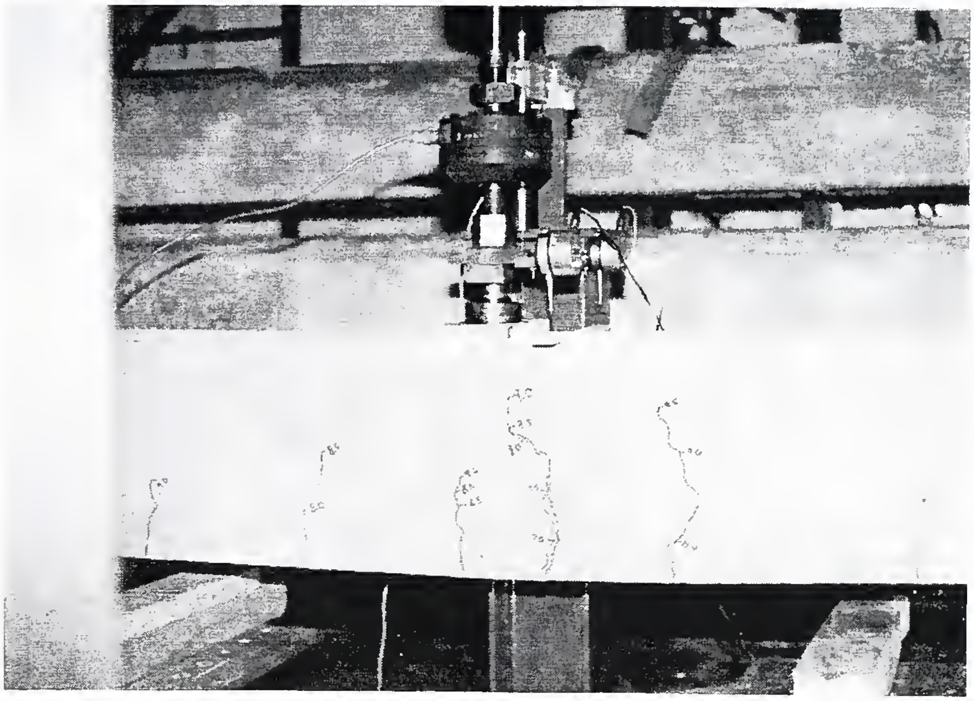
7SLW-FWC-3L



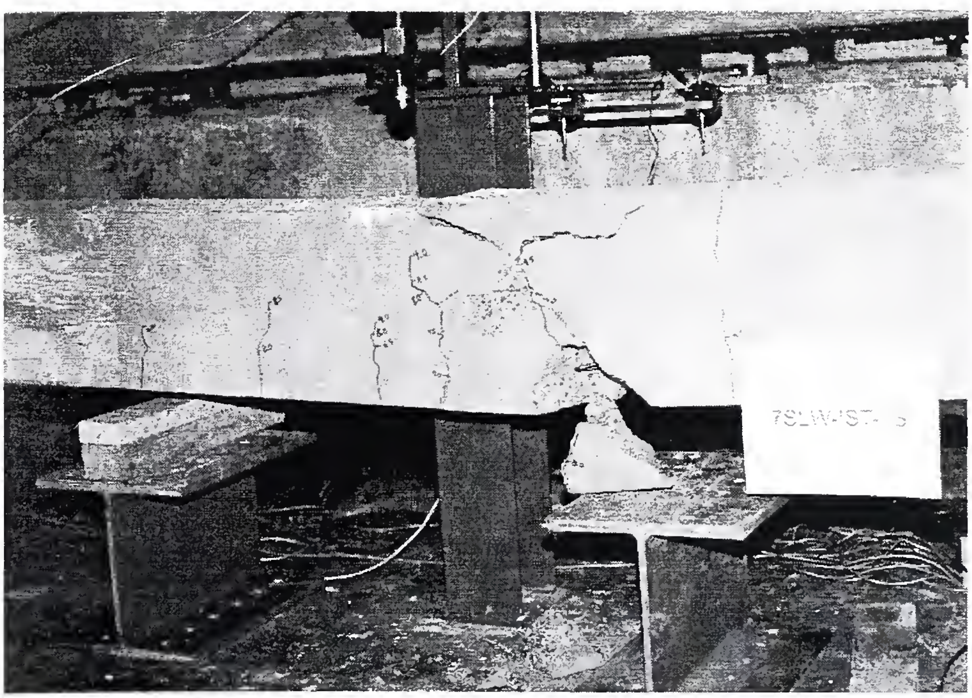
7SLW-IST-1S



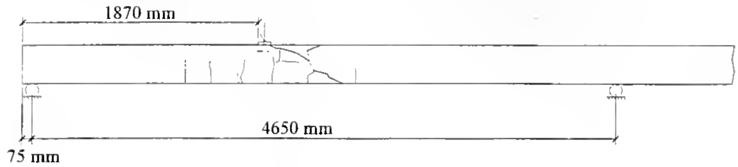
7SLW-IST-1S – Cracks at Failure



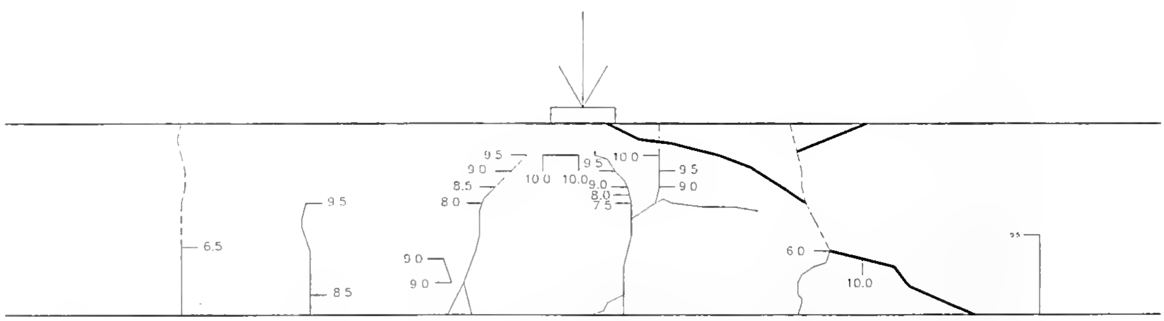
7SLW-IST-1S



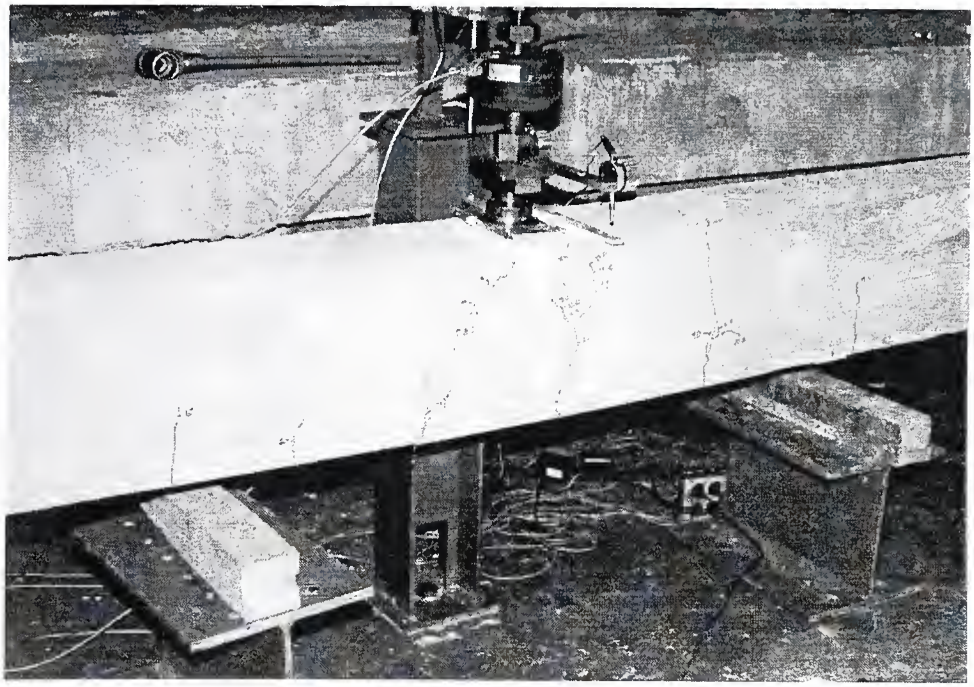
7SLW-IST-1S



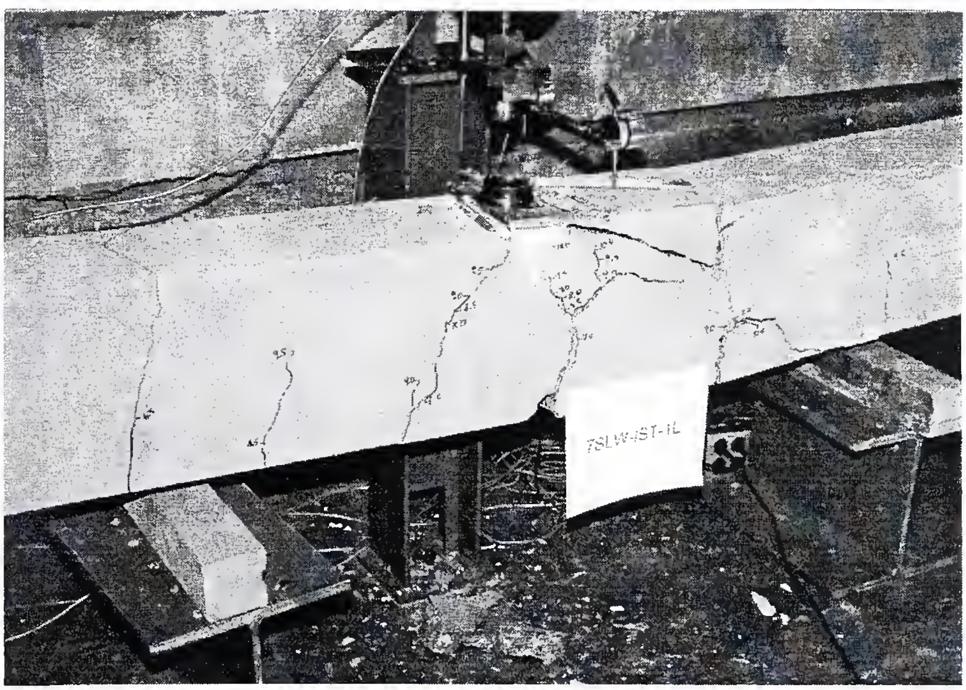
7SLW-IST-1L



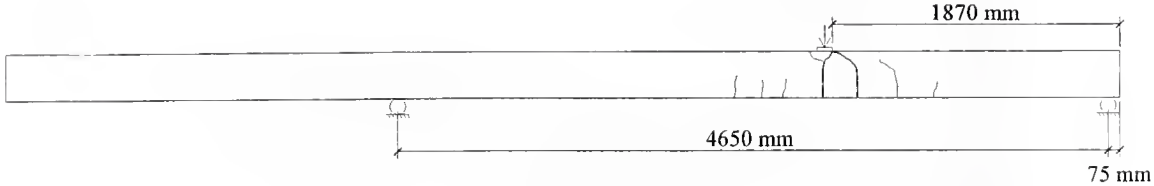
7SLW-IST-1L – Cracks at Failure



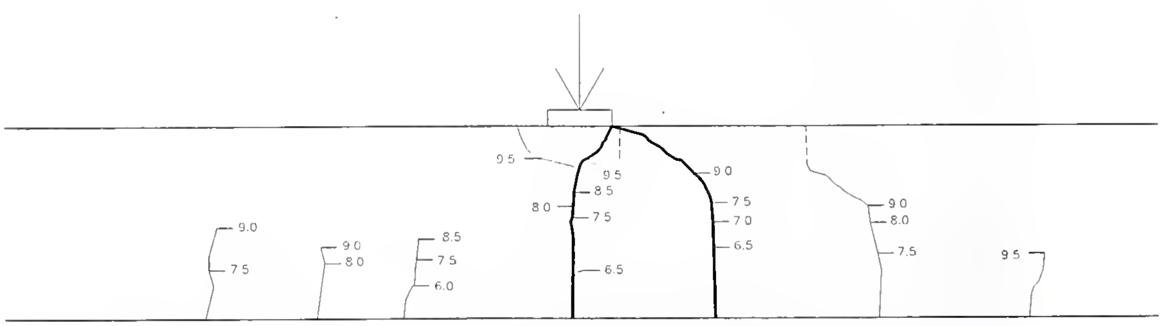
7SLW-IST-1L



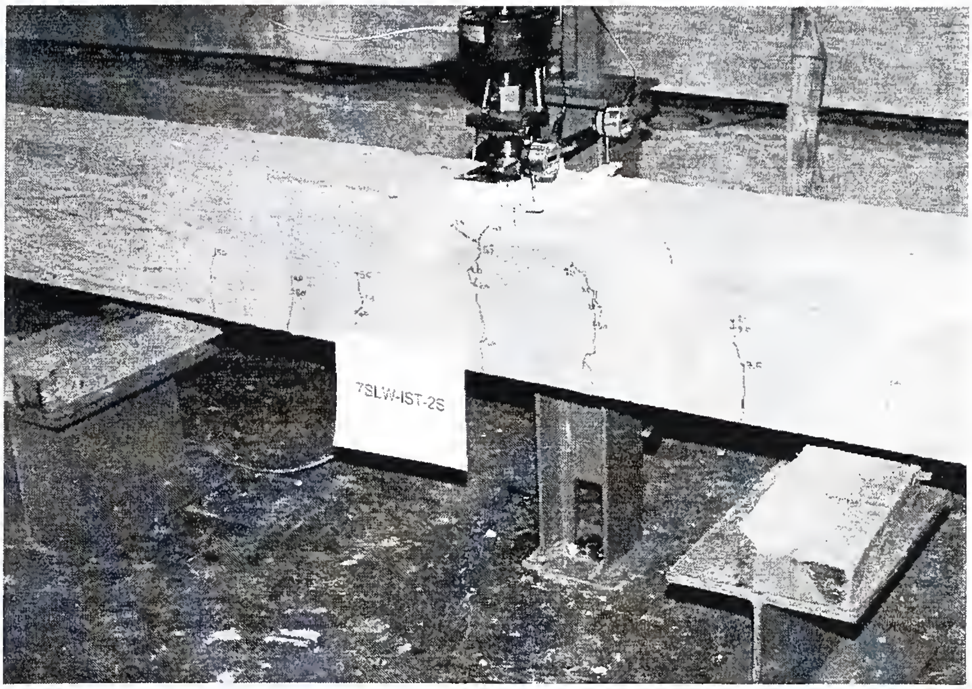
7SLW-IST-1L



7SLW-IST-2S

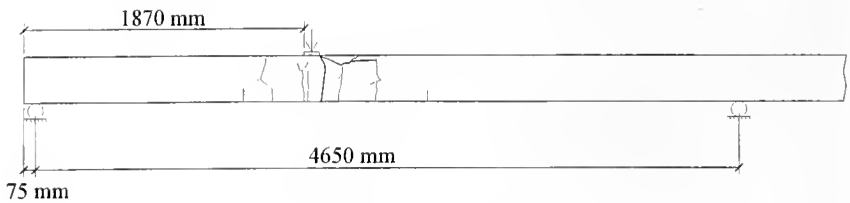


7SLW-IST-2S – Cracks at Failure

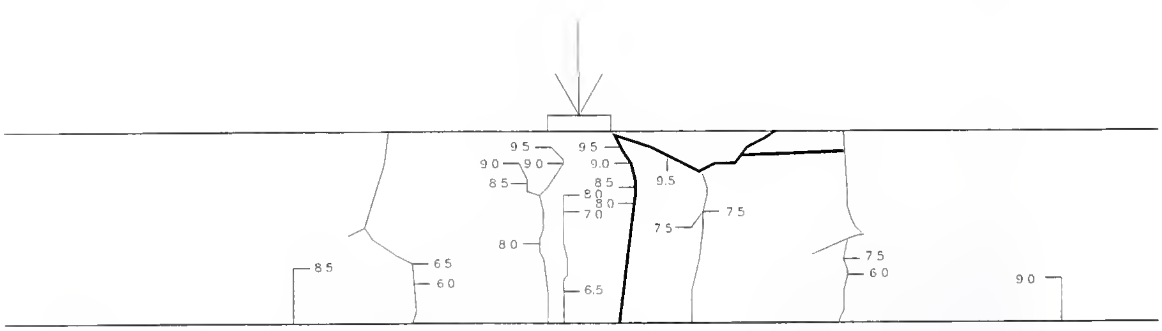


7SLW-IST-2S

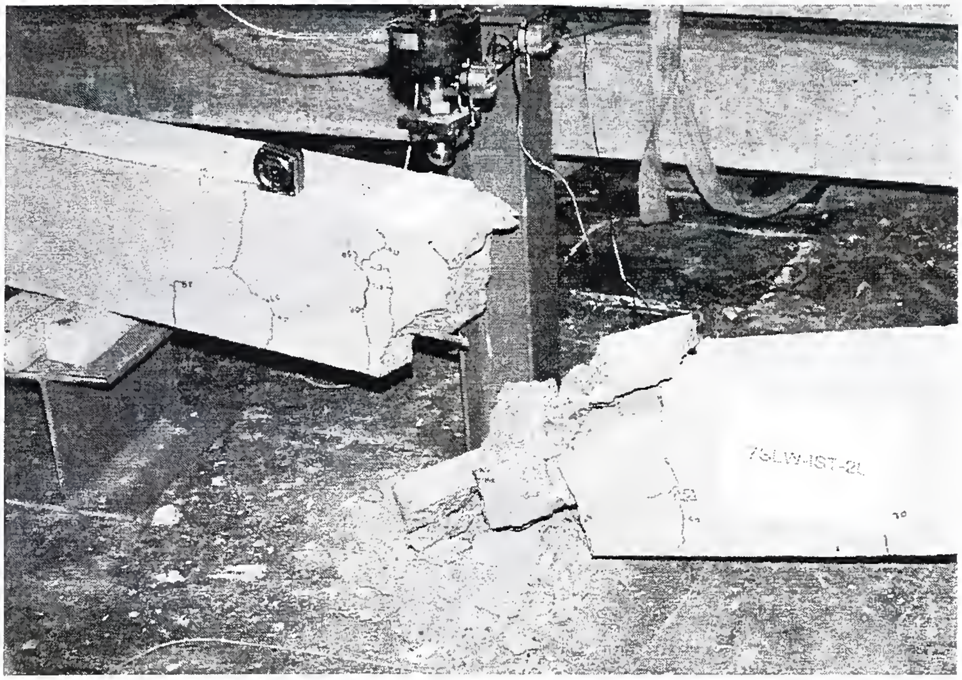
7SLW-IST-2S



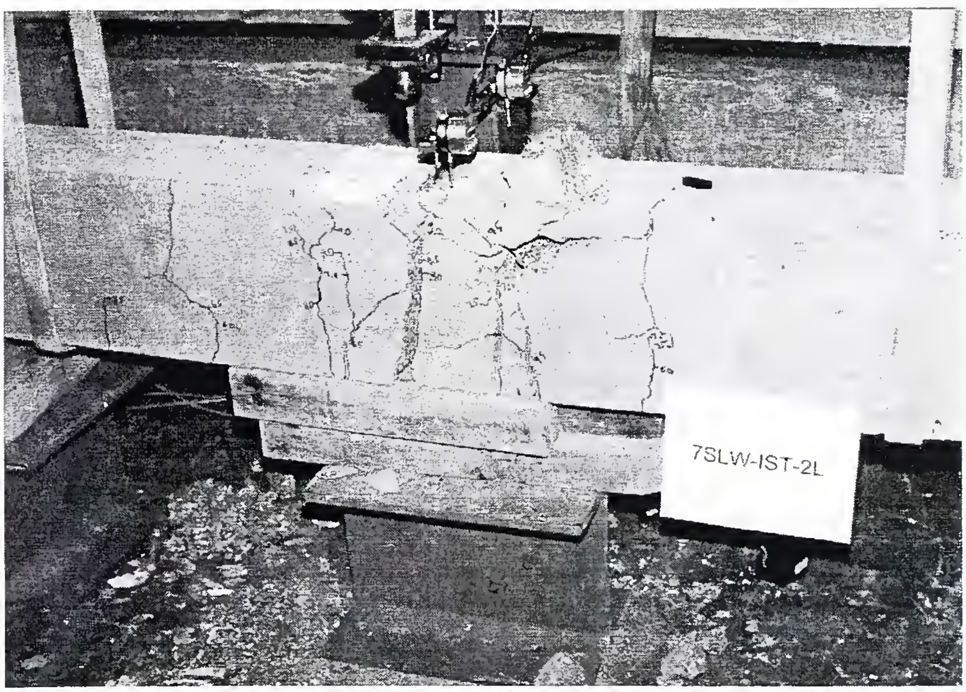
7SLW-IST-2L



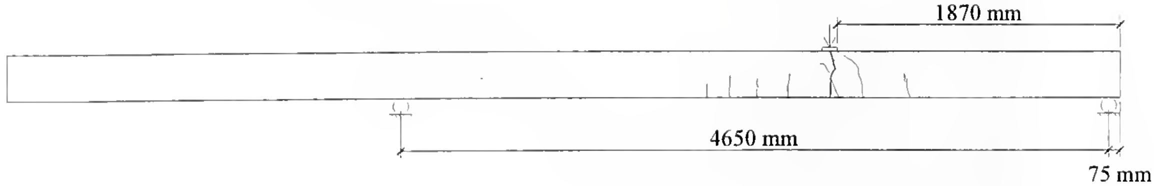
7SLW-IST-2L – Cracks at Failure



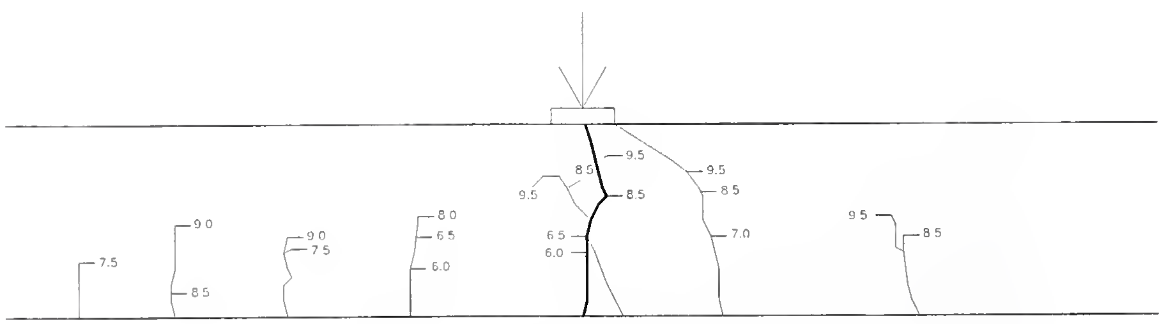
7SLW-IST-2L



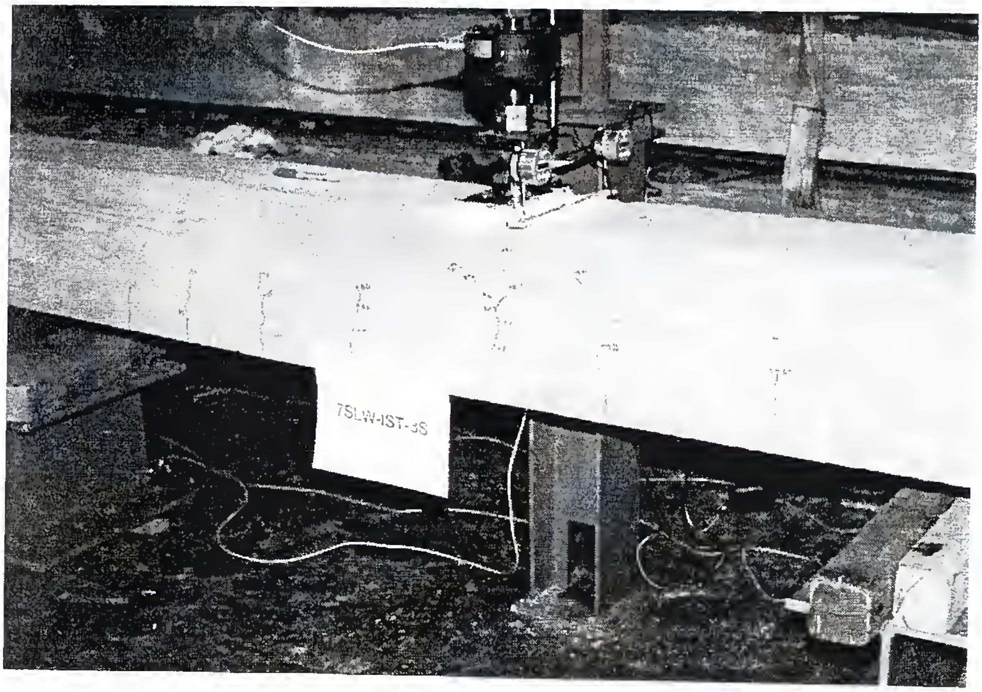
7SLW-IST-2L



7SLW-IST-3S



7SLW-IST-3S – Cracks at Failure



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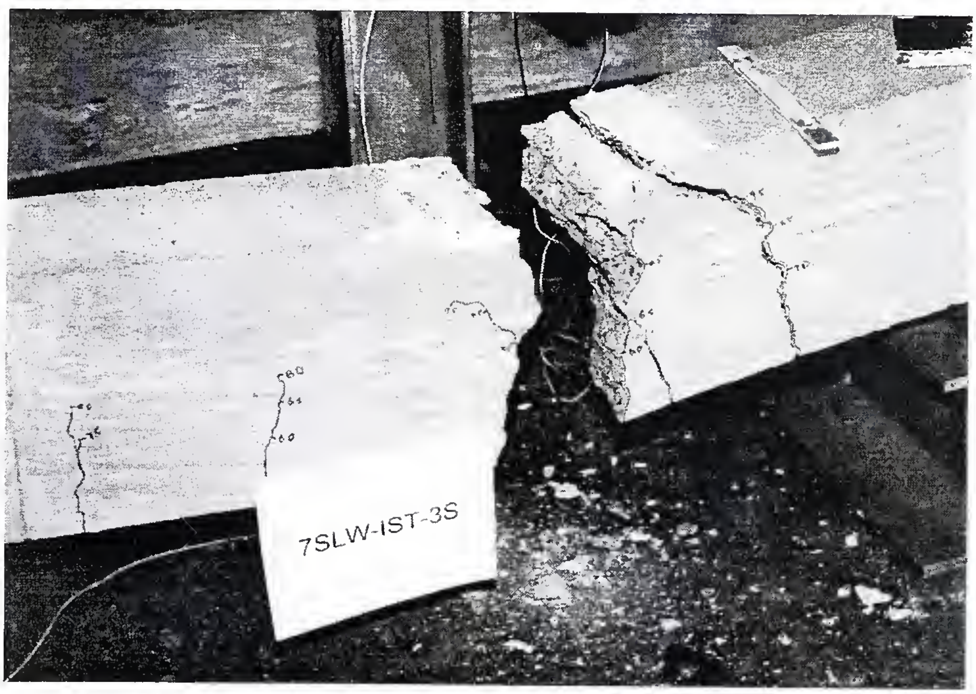

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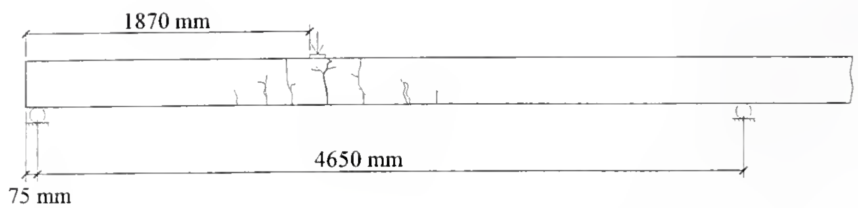

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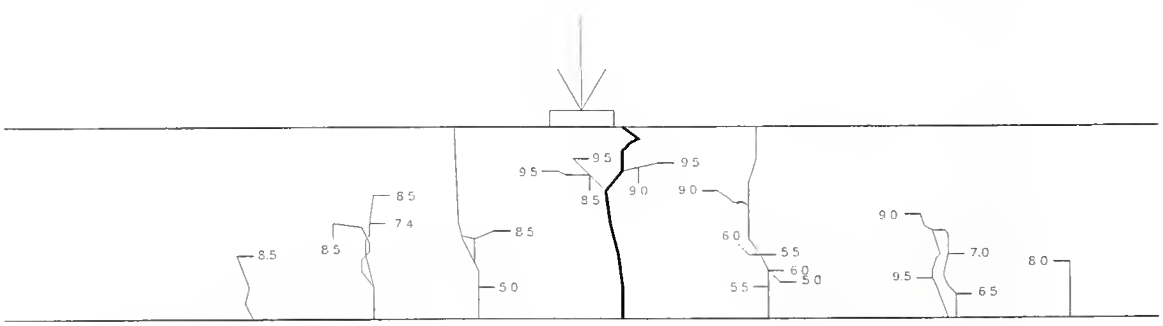

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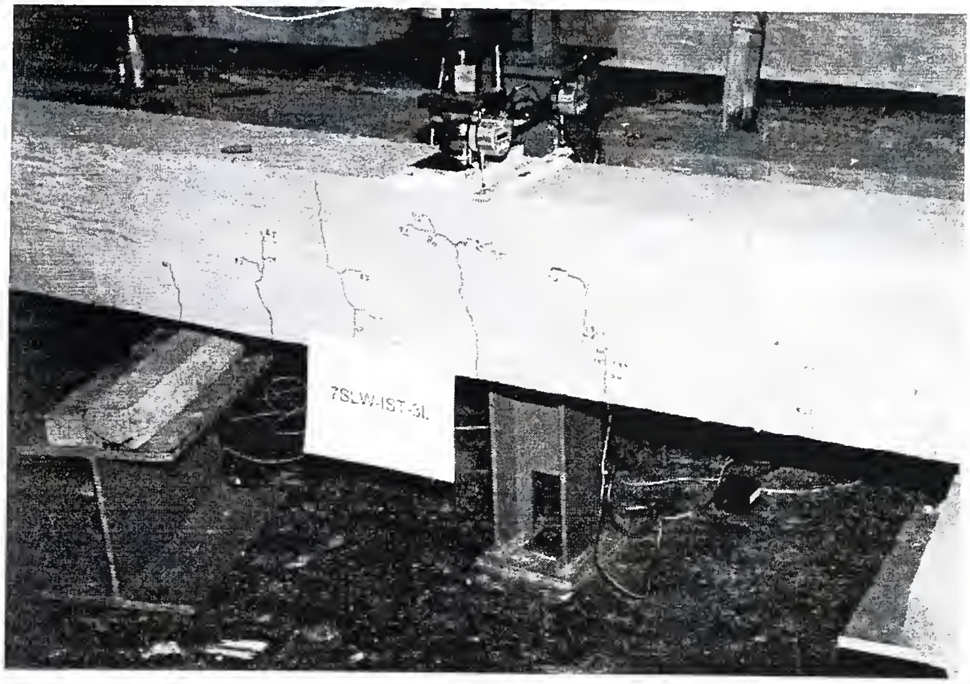
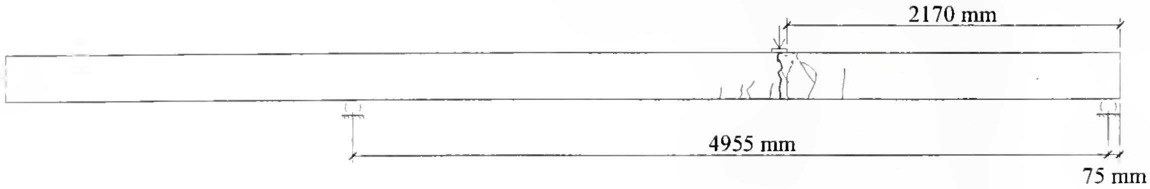


7SLW-IST-3L

7SLW-IST-3L



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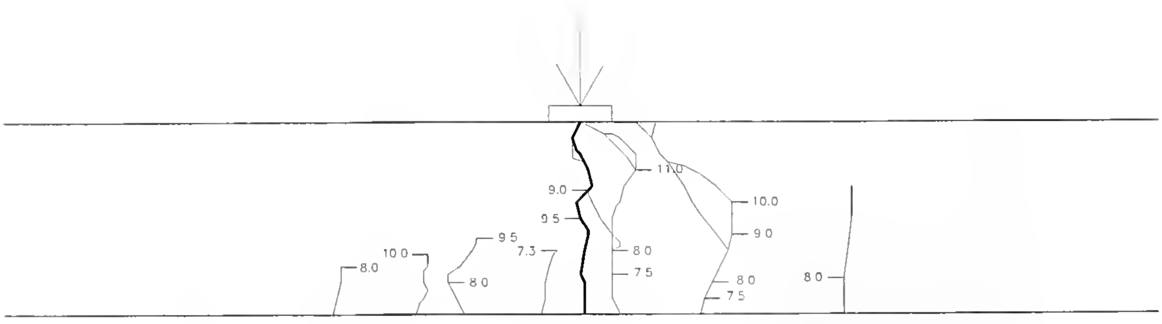

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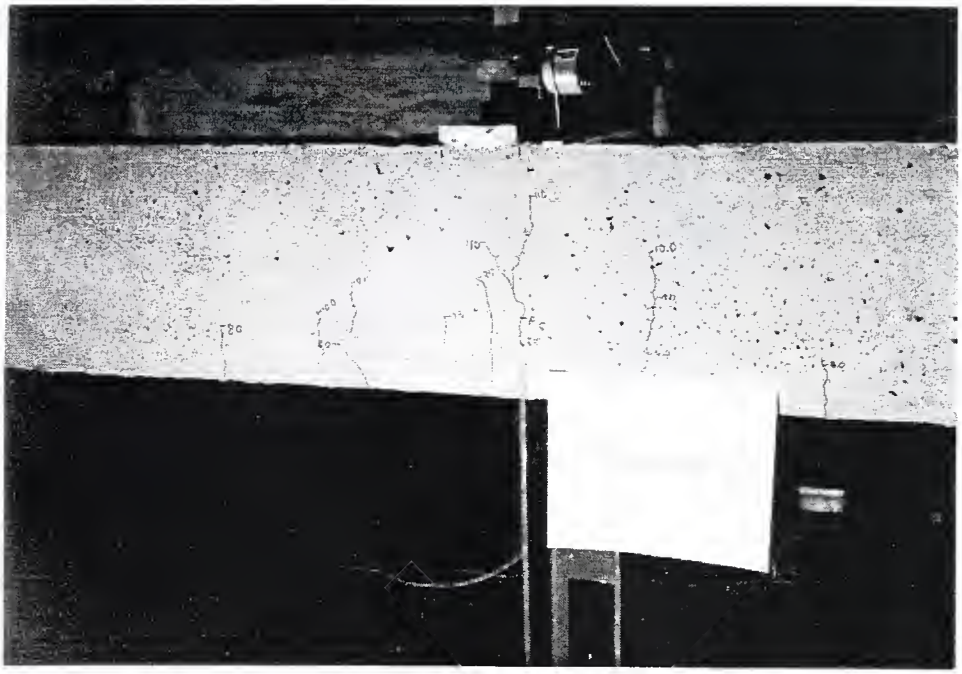

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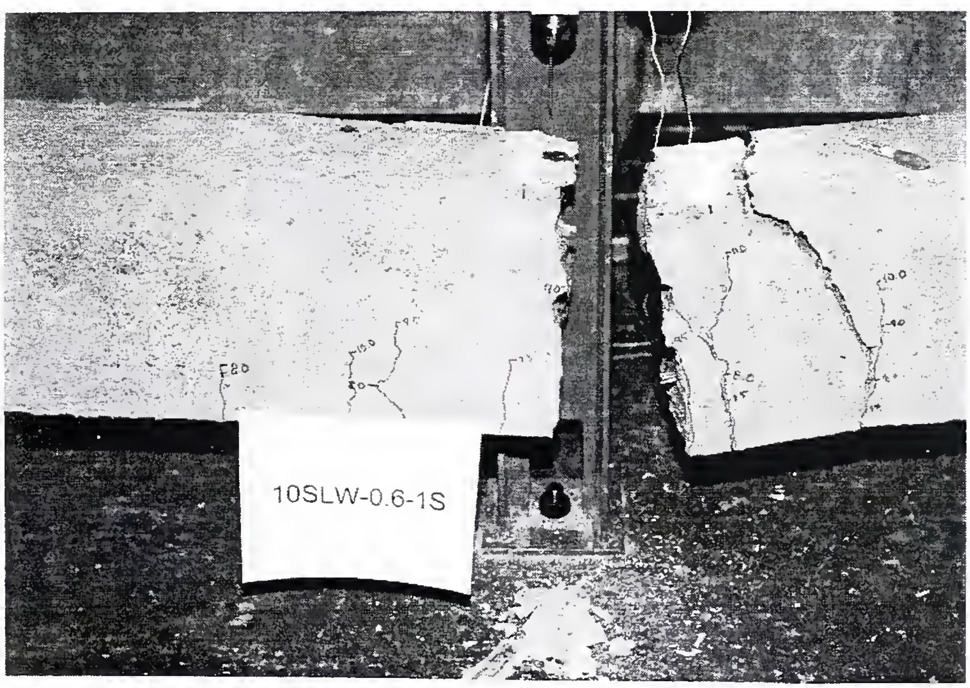

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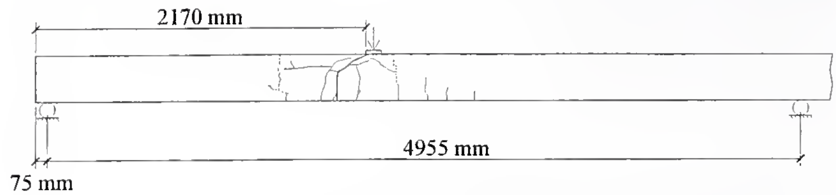

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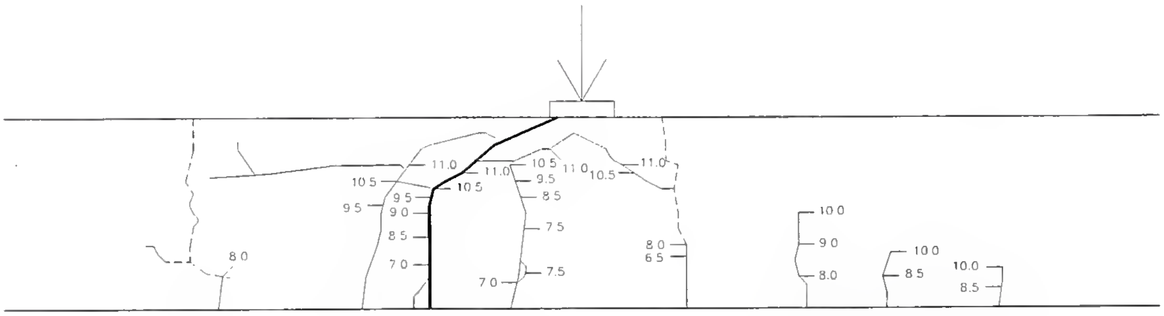

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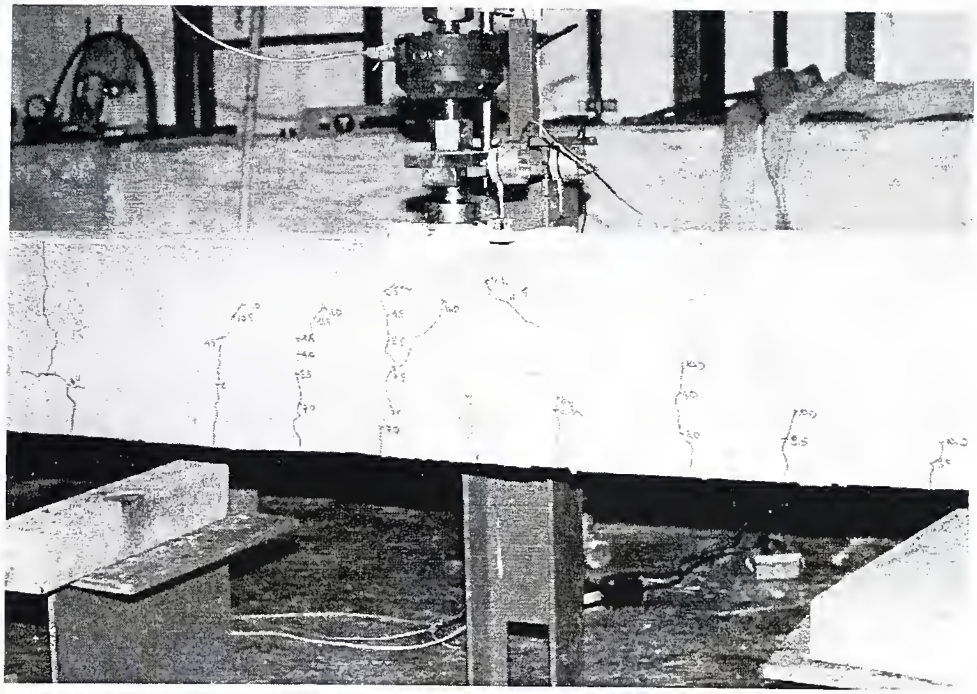

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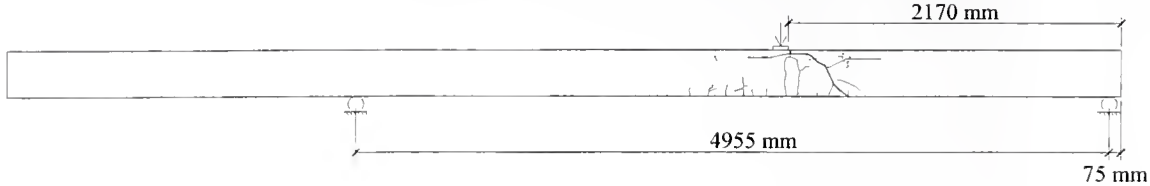

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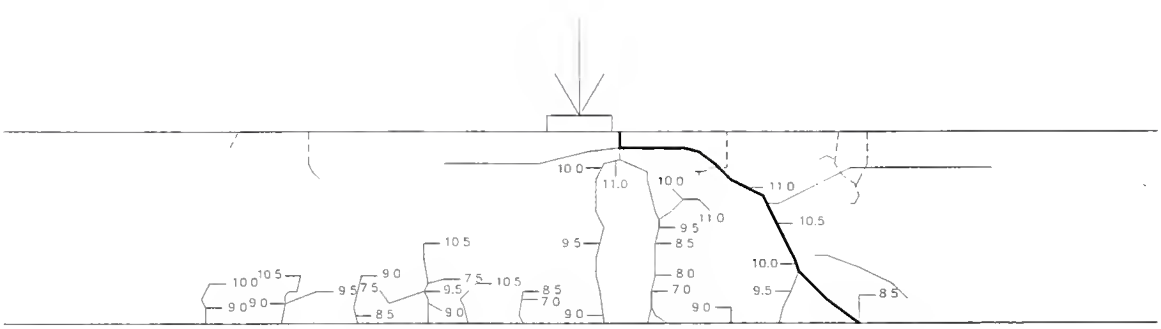

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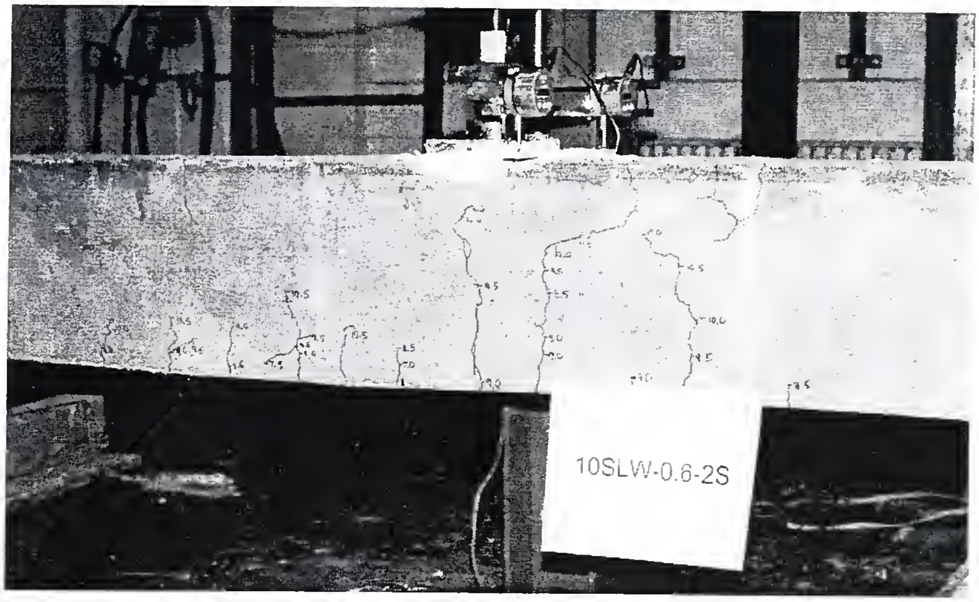

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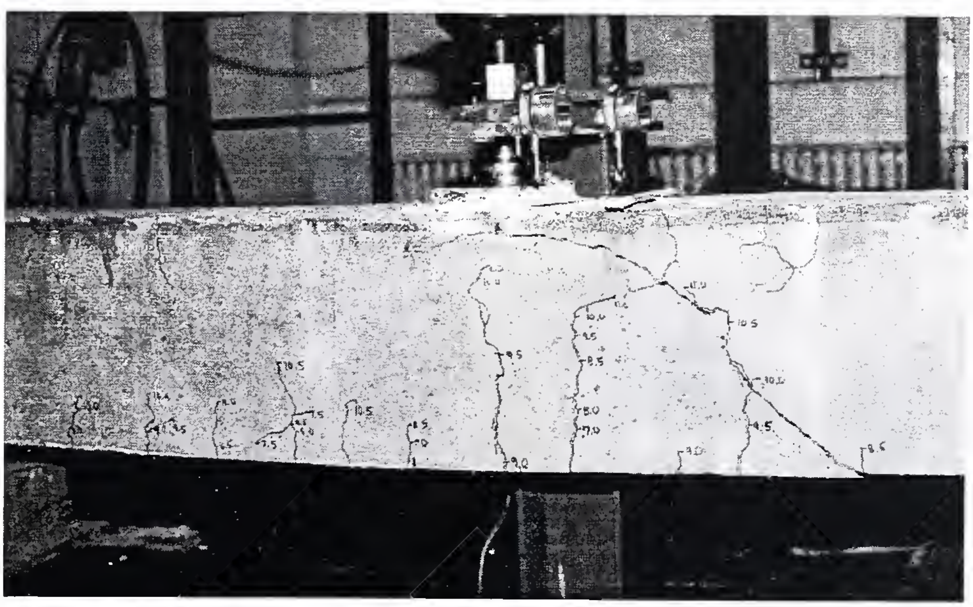

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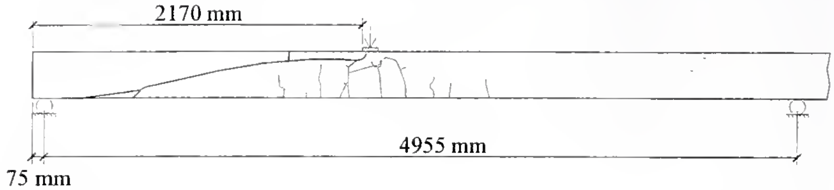

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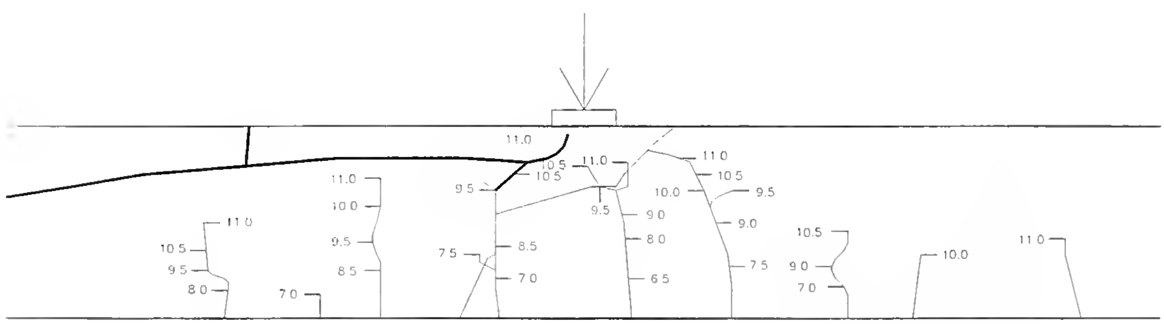

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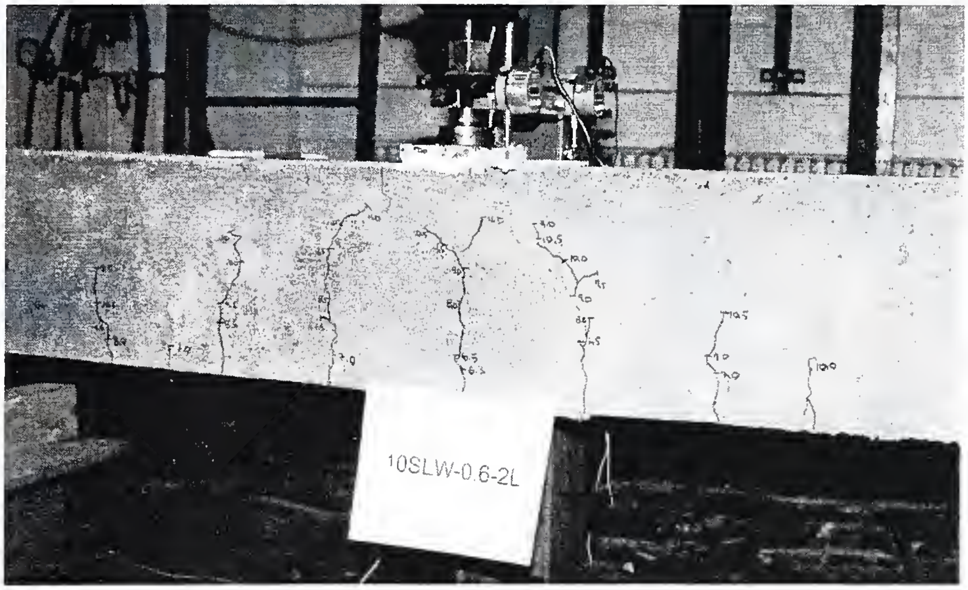

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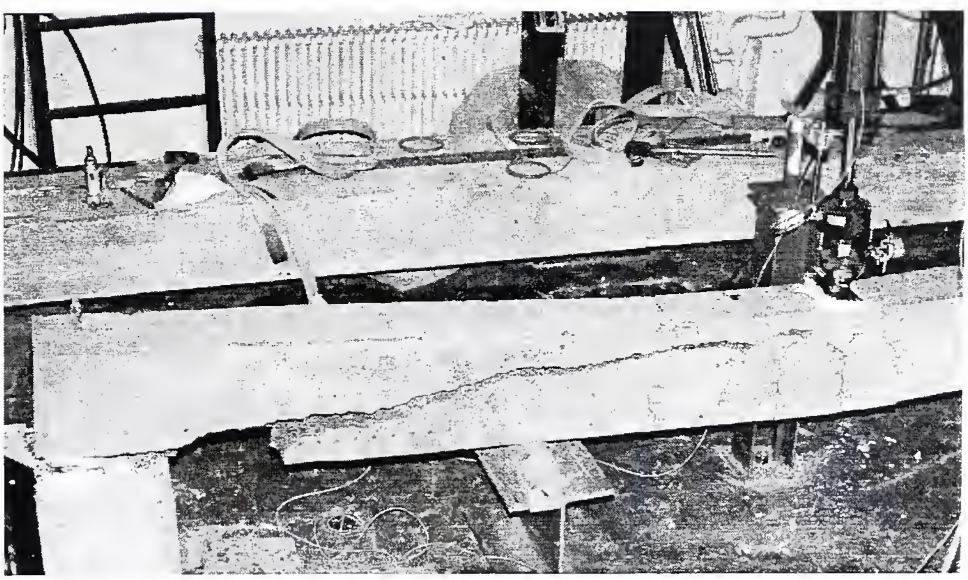

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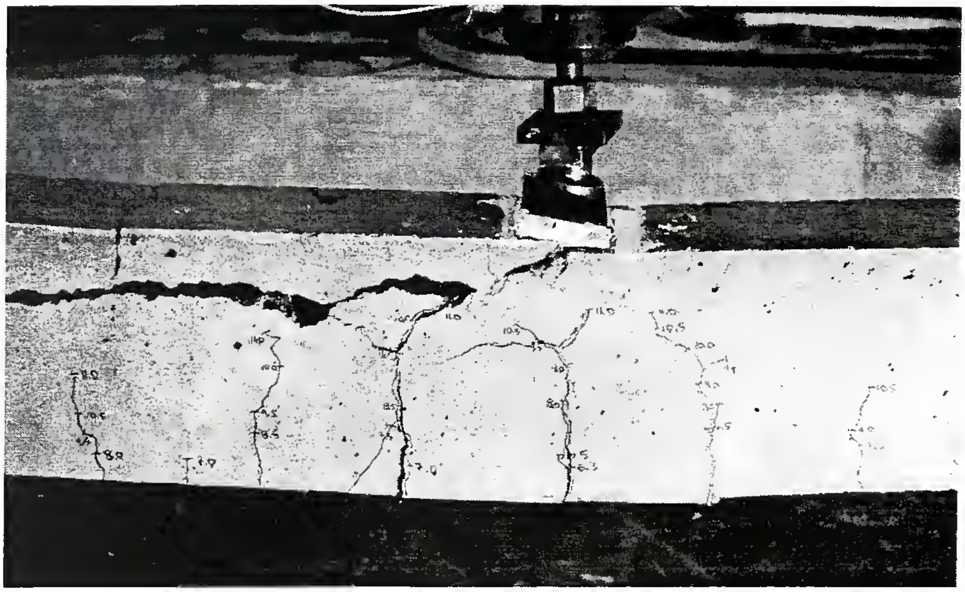

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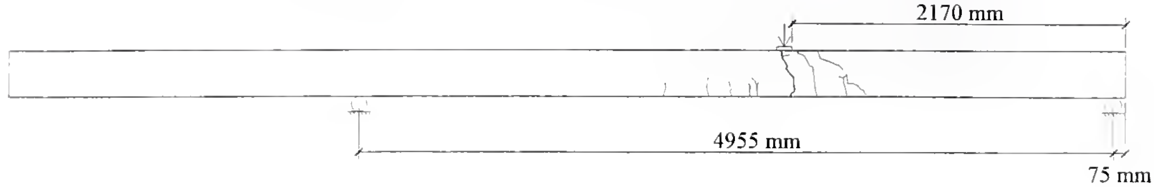

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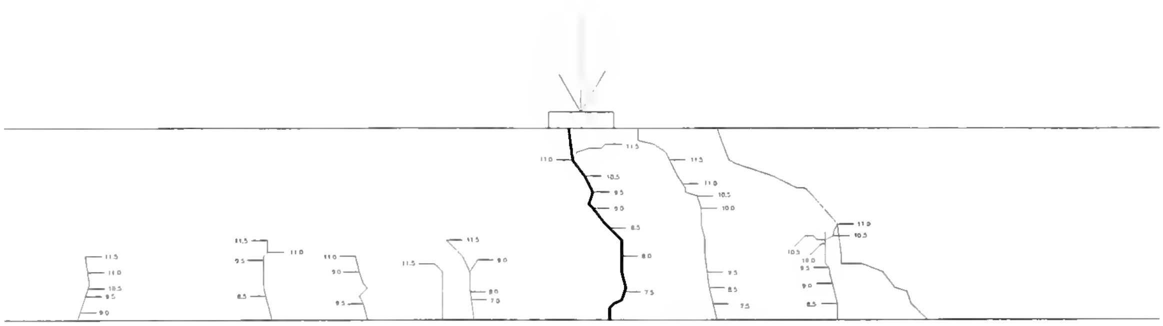

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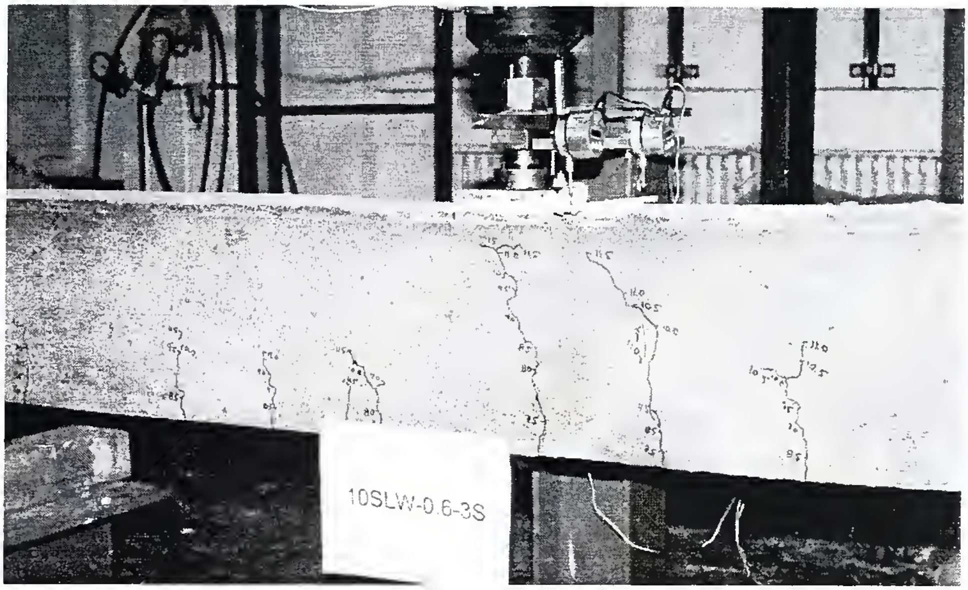

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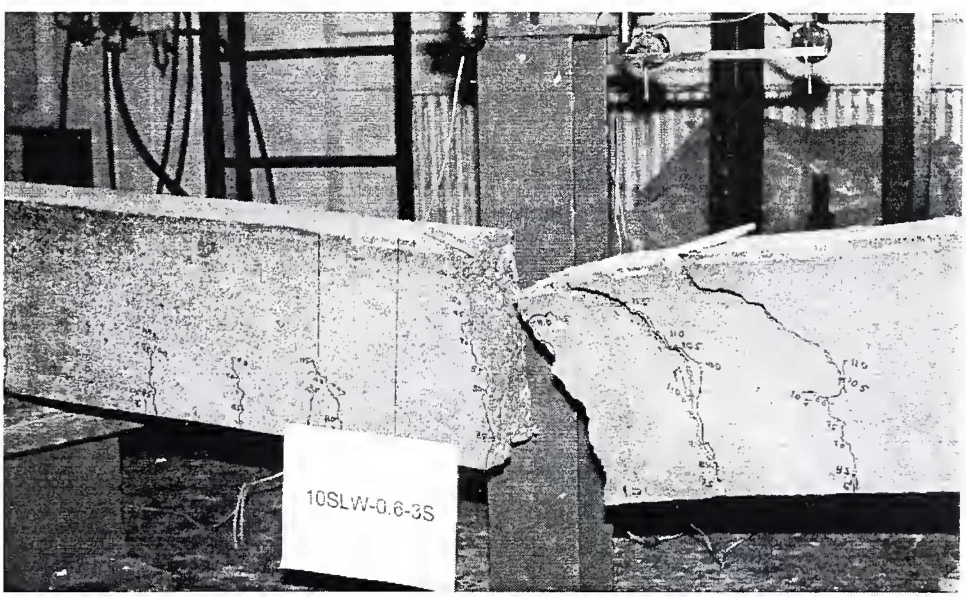

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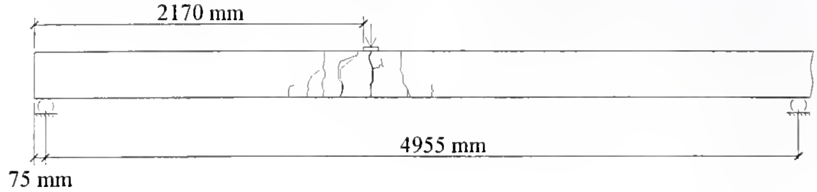

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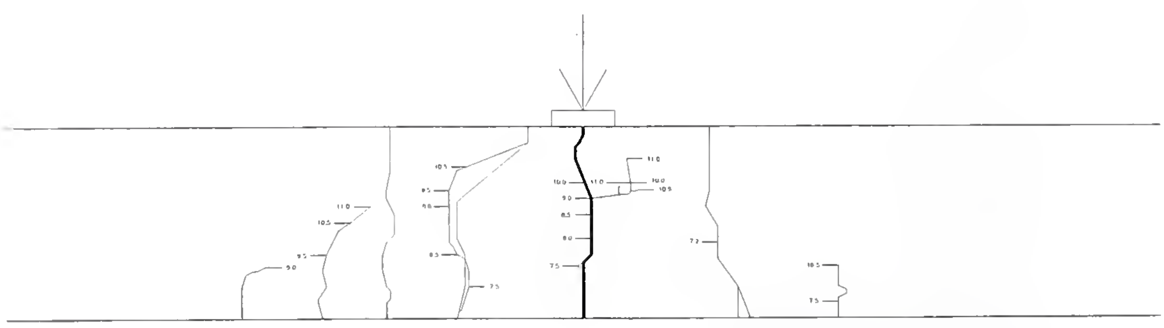

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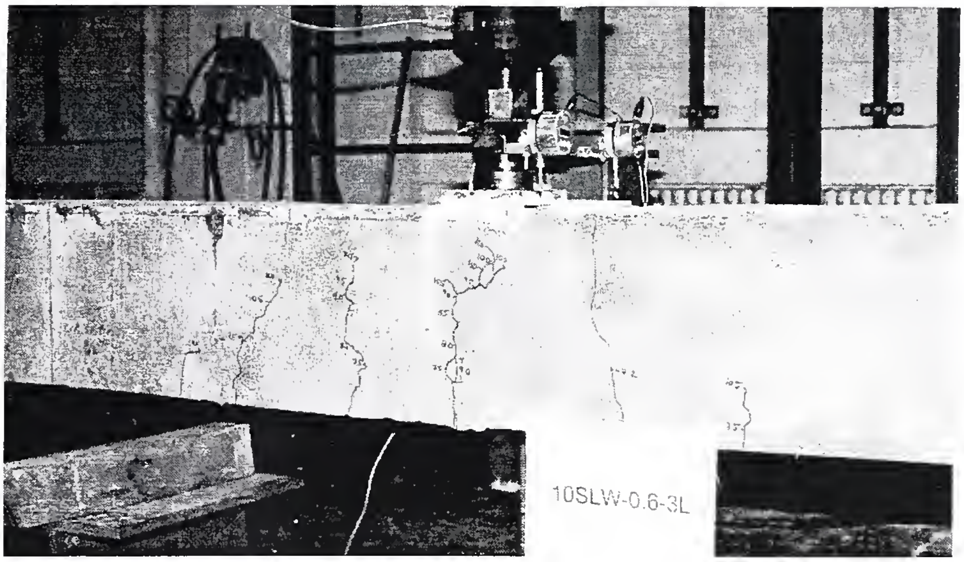

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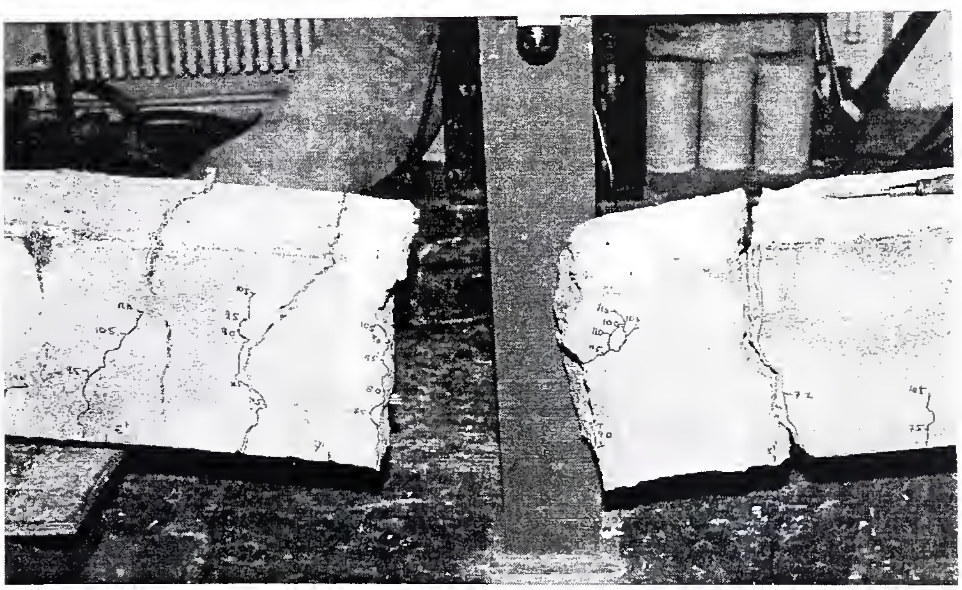
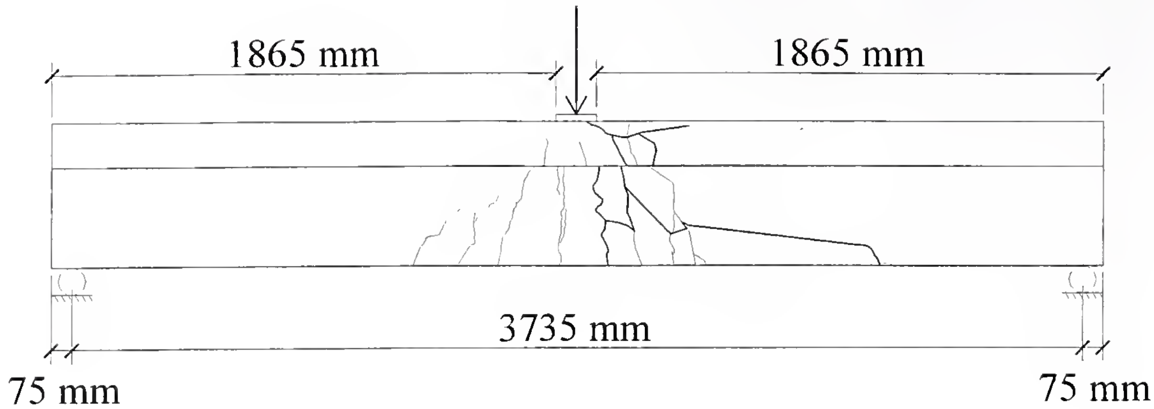


10SLW-0.6-3L

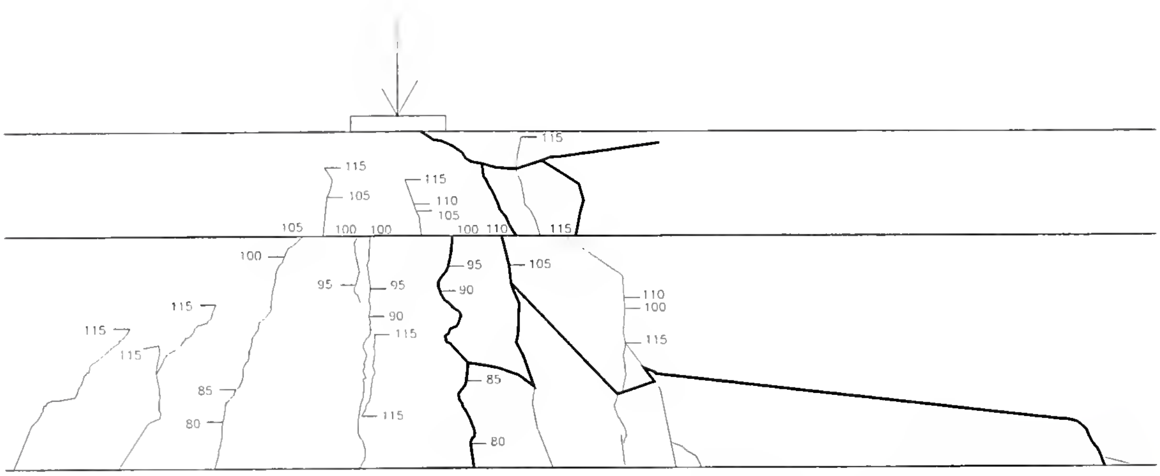
APPENDIX E

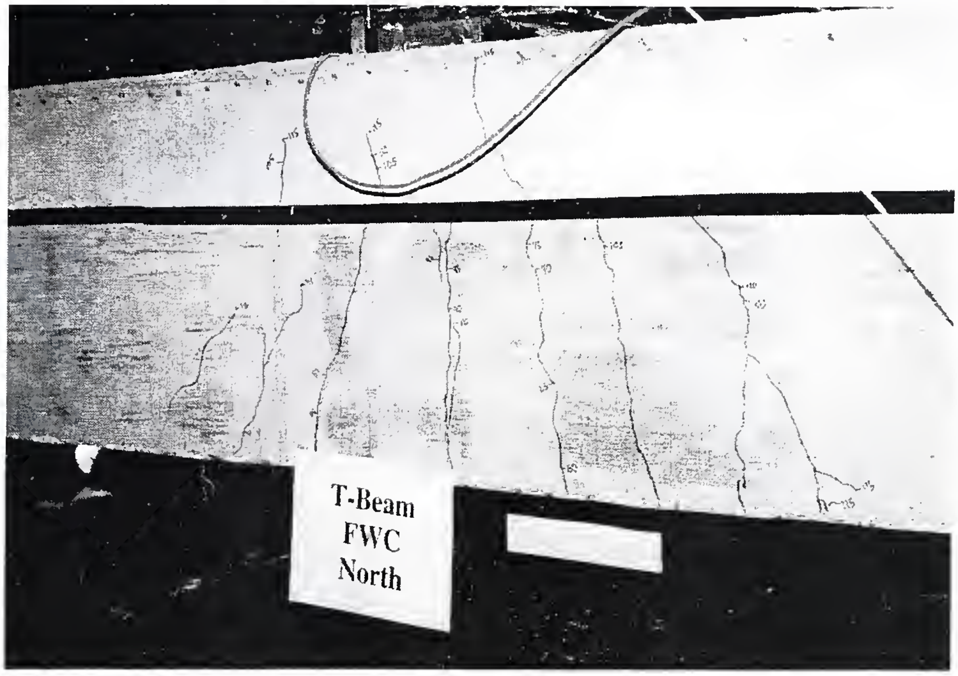
T-BEAM TESTS -- CRACK PATTERNS AND PHOTOS



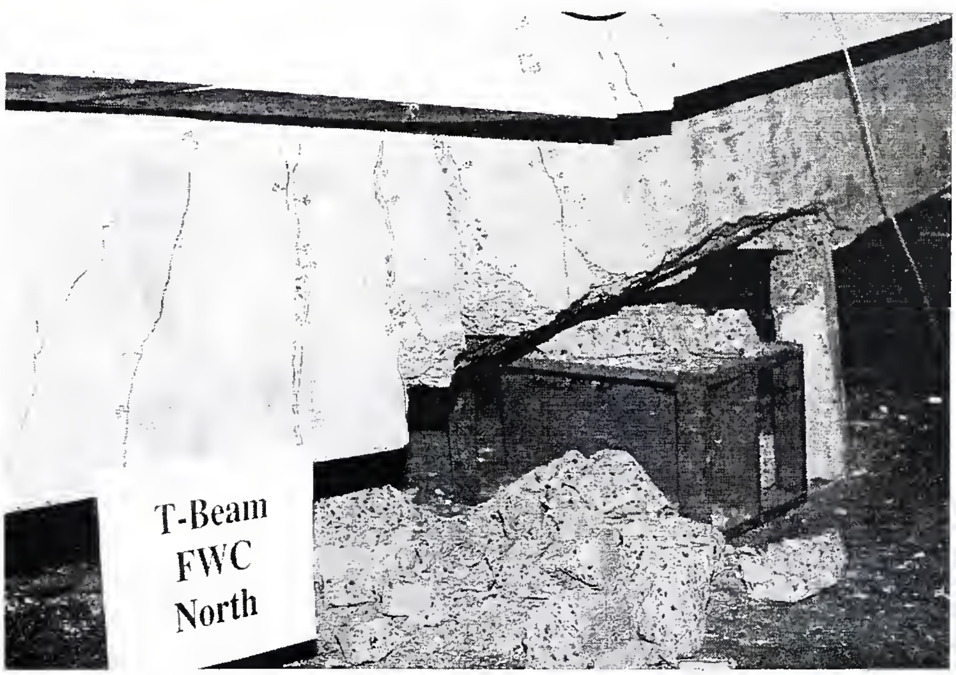
T-Beam FWC

Crack Patterns on North Face





T-Beam FWC (North Face)



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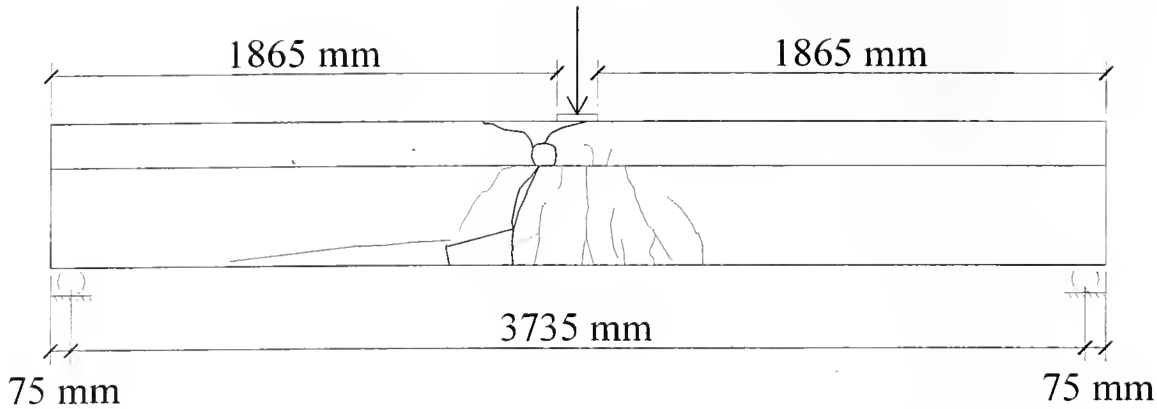
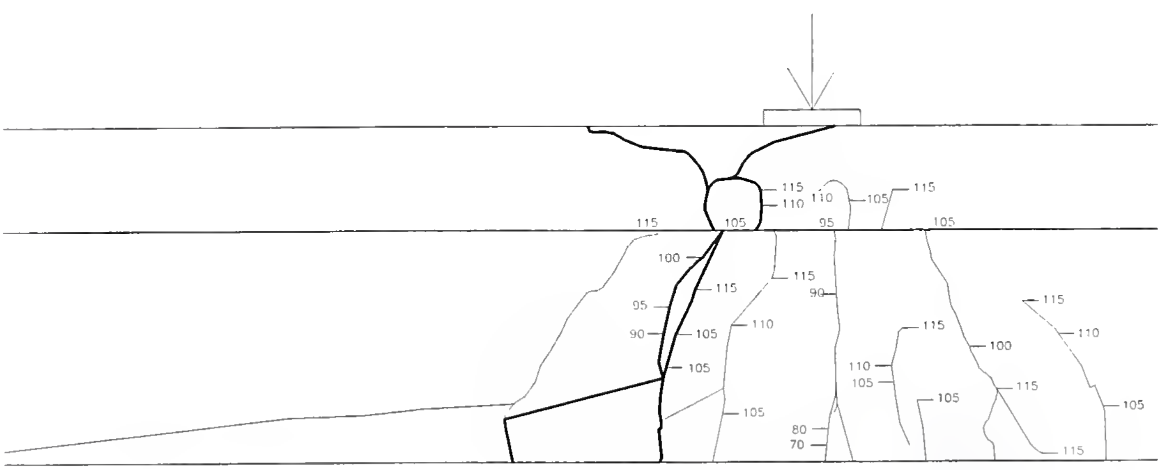
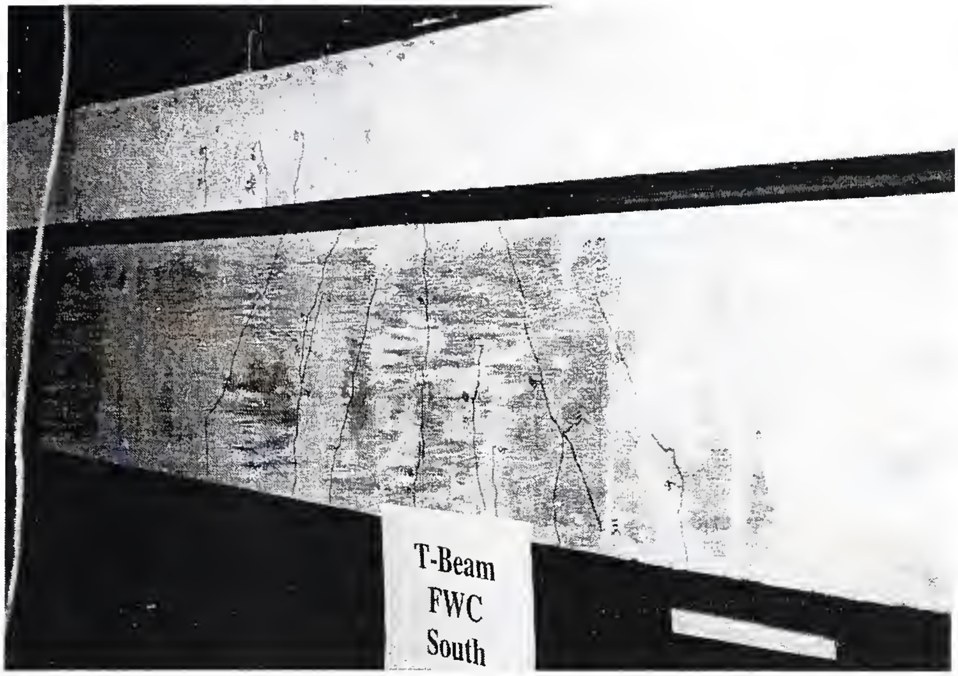


T-Beam FWC

Crack Patterns on South Face





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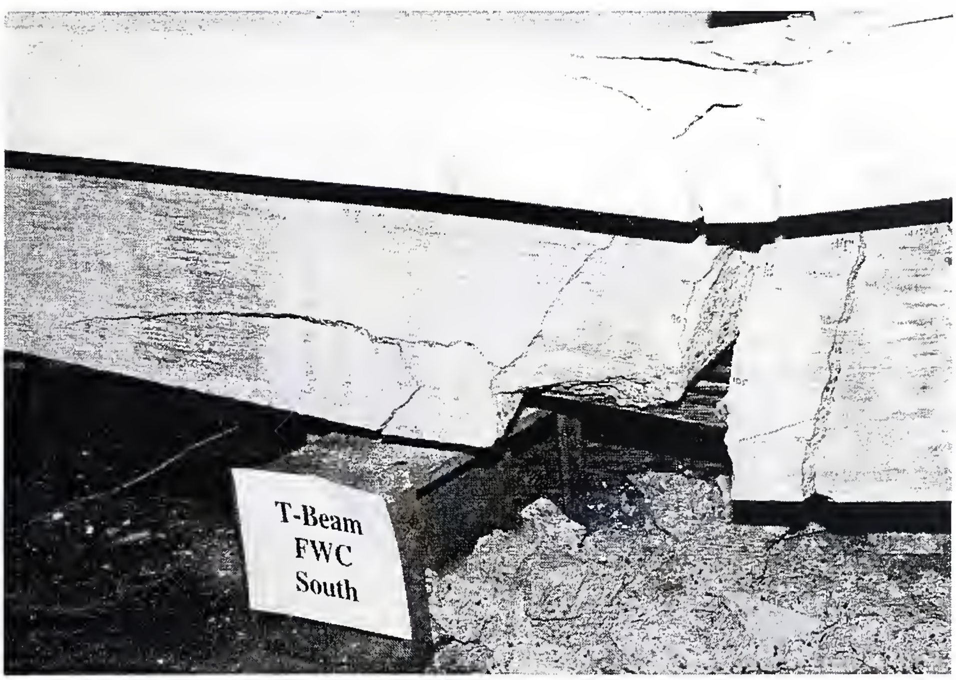

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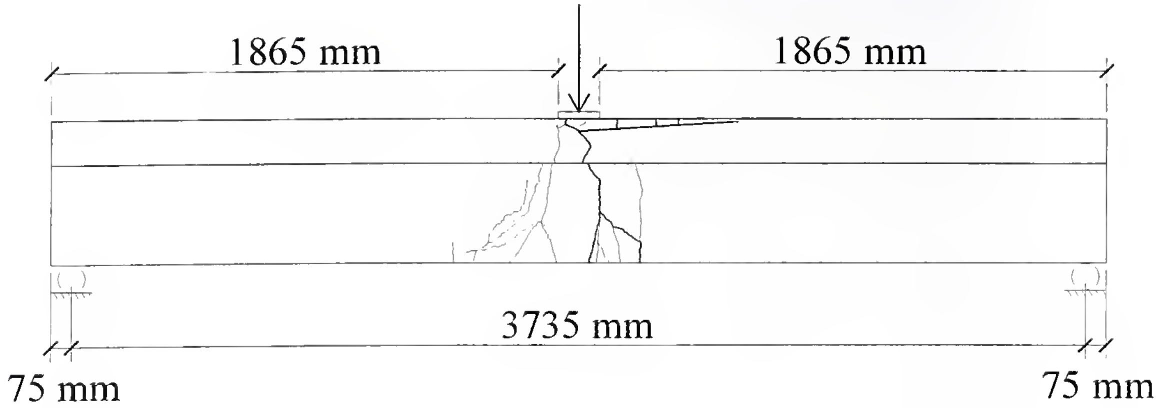
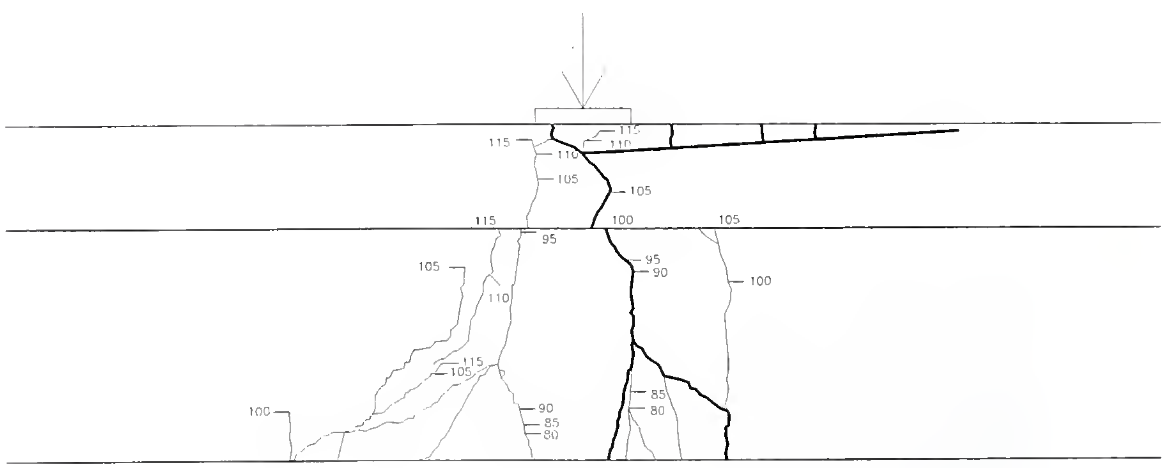
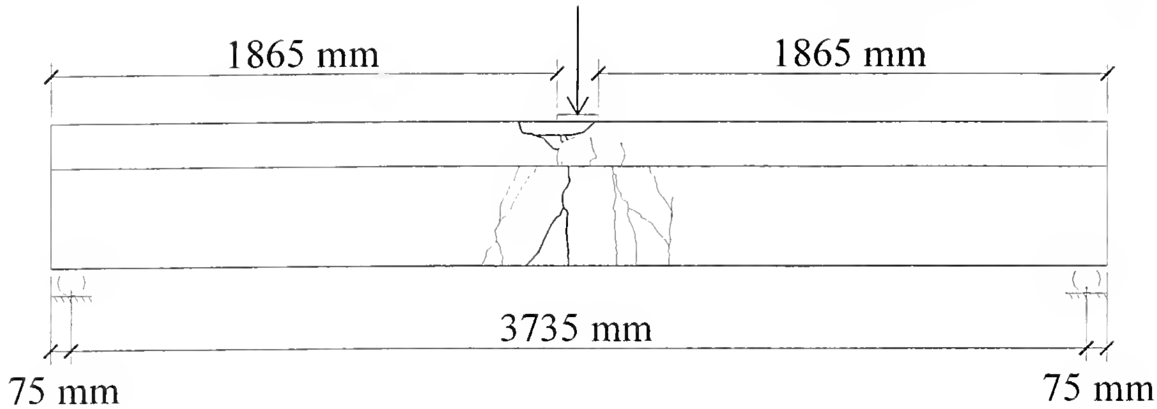


T-Beam IST1

Crack Patterns on North Face

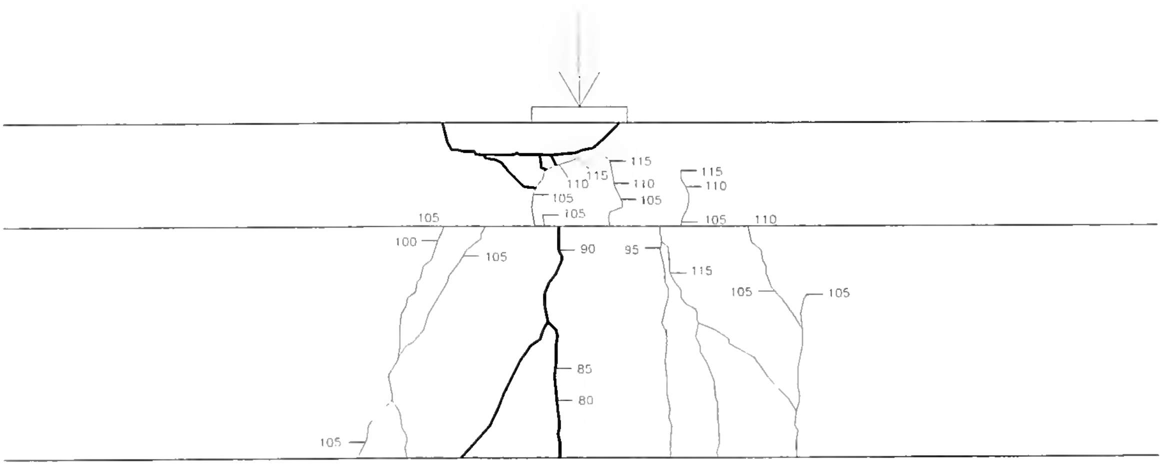


No photos available

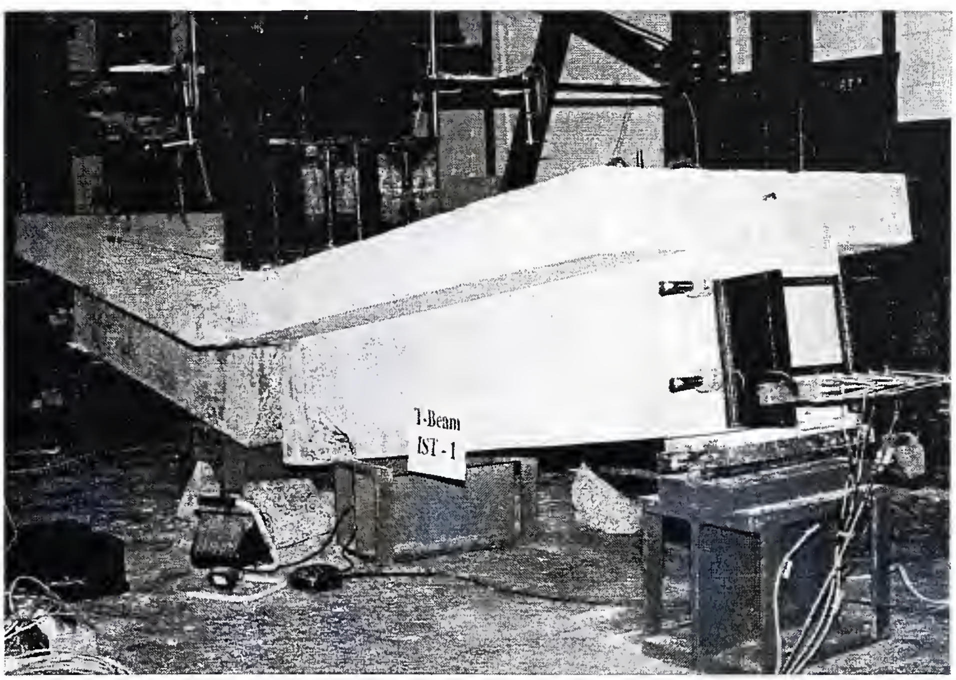


T-Beam IST1

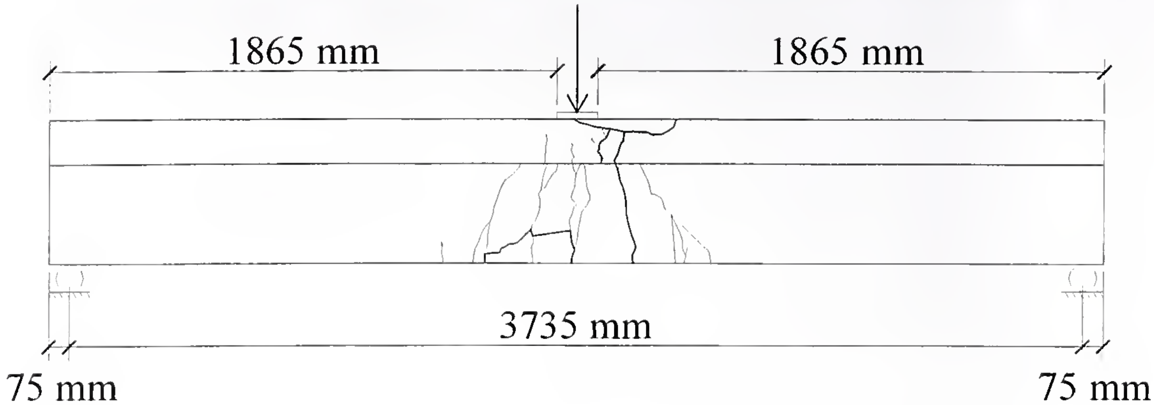
Crack Patterns on South Face



T-Beam IST1 (South Face)

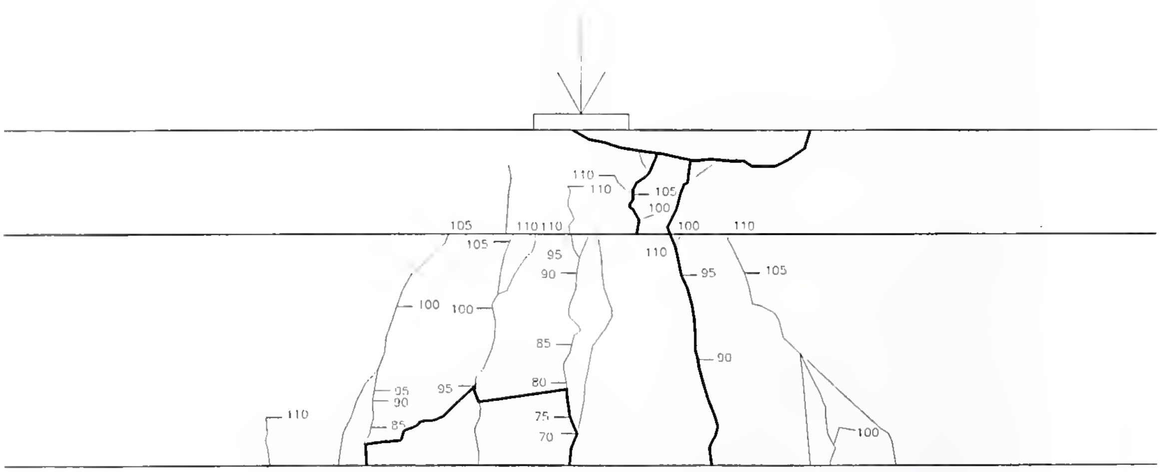


T-Beam IST1 (South Face)



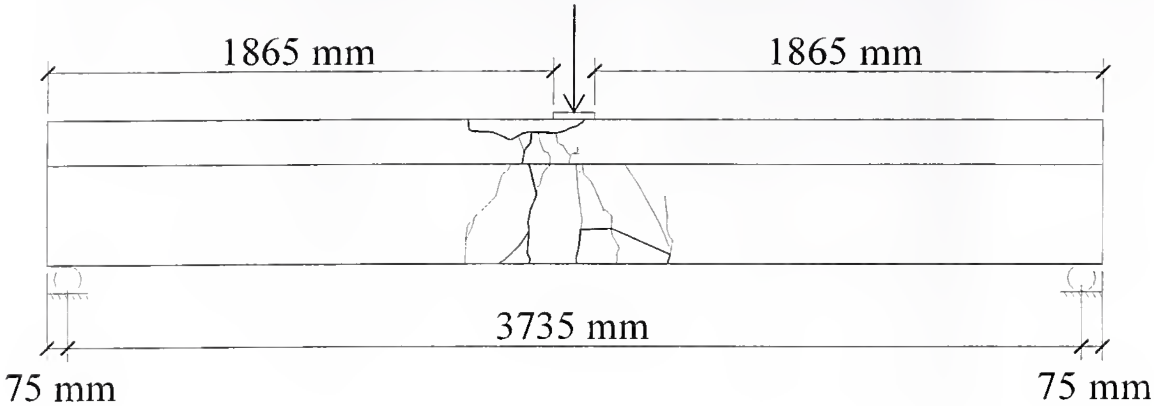
T-Beam IST2

Crack Patterns on North Face



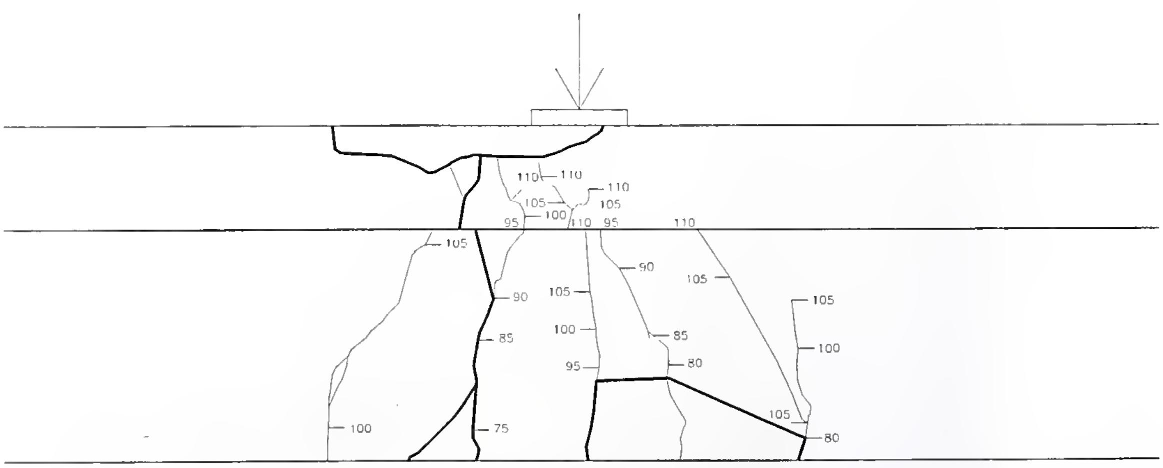
T-Beam IST2 (North Face)

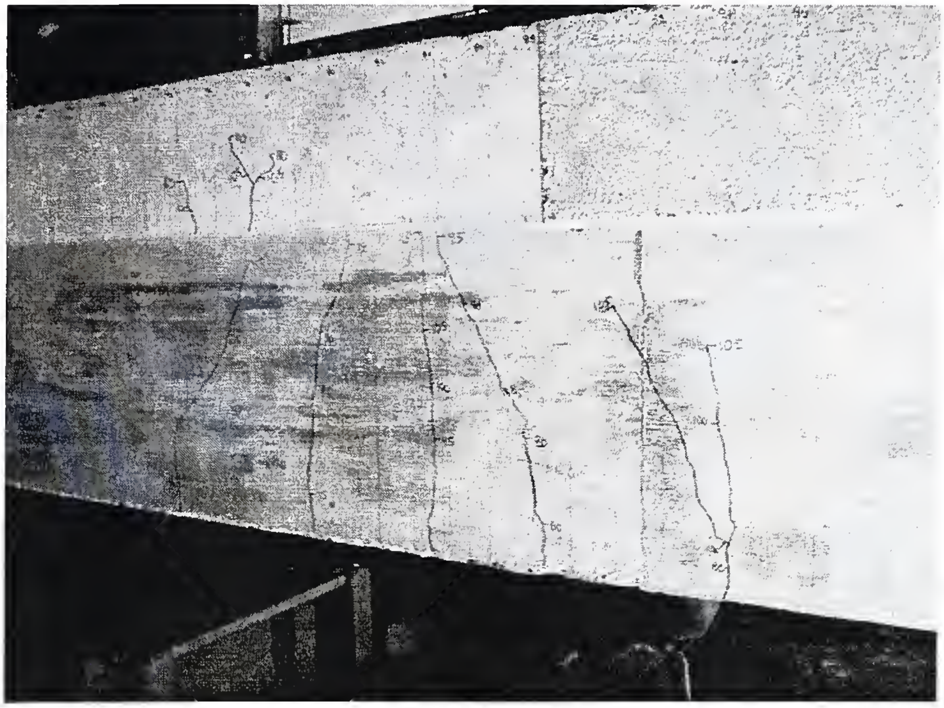
T-Beam IST2 (North Face)



T-Beam IST2

Crack Patterns on South Face





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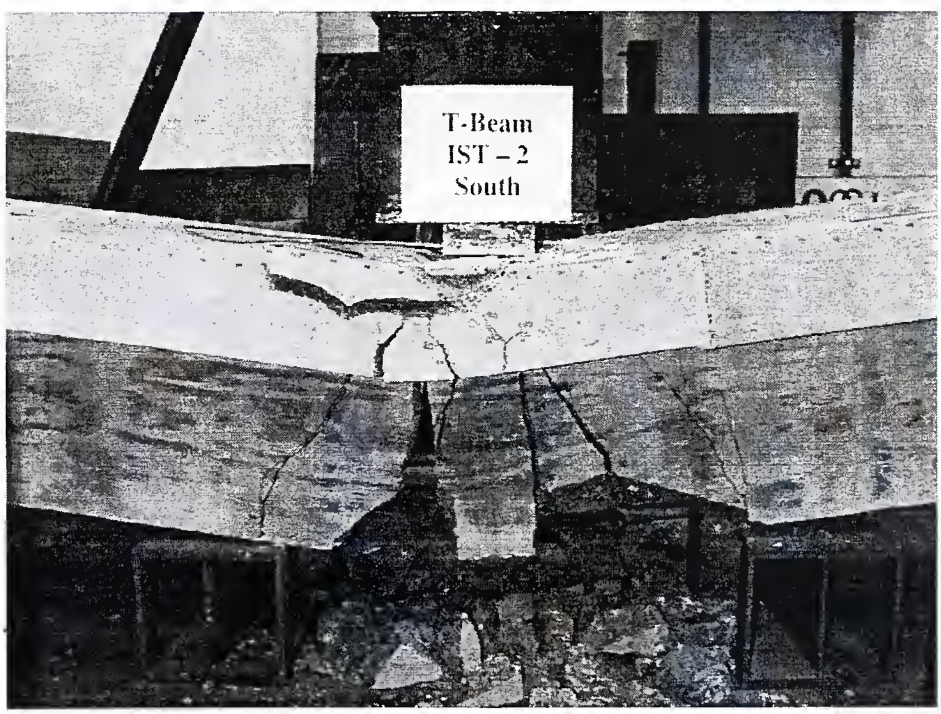
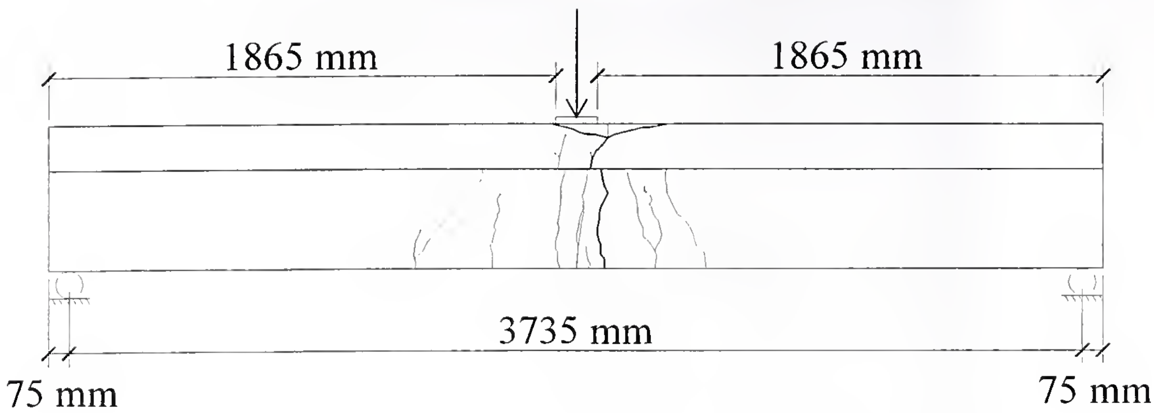
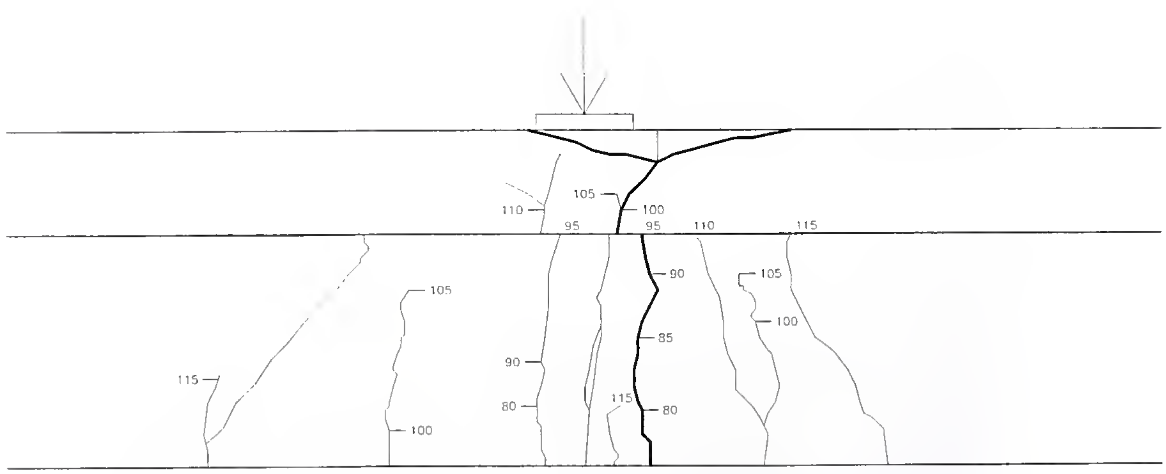


T-Beam IST2 (South Face)



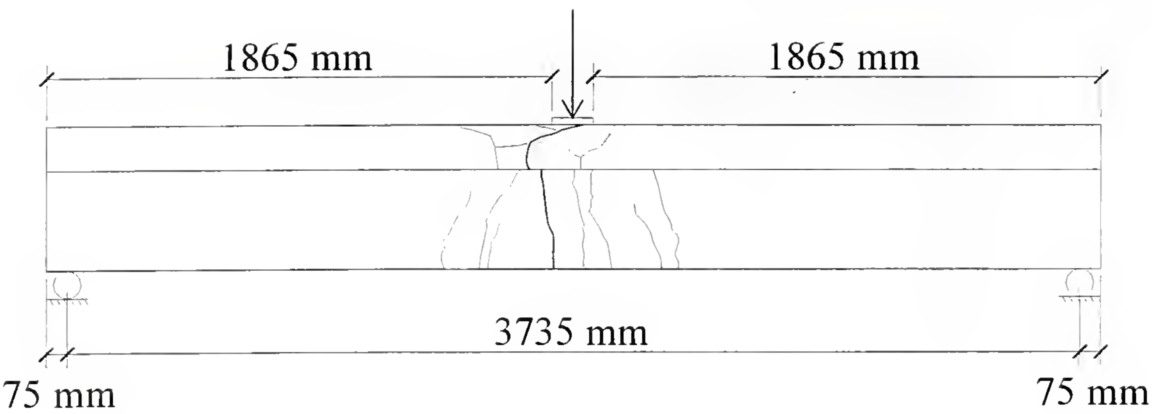
T-Beam FWC - 3" Stirrup Spacing

Crack Patterns on North Face



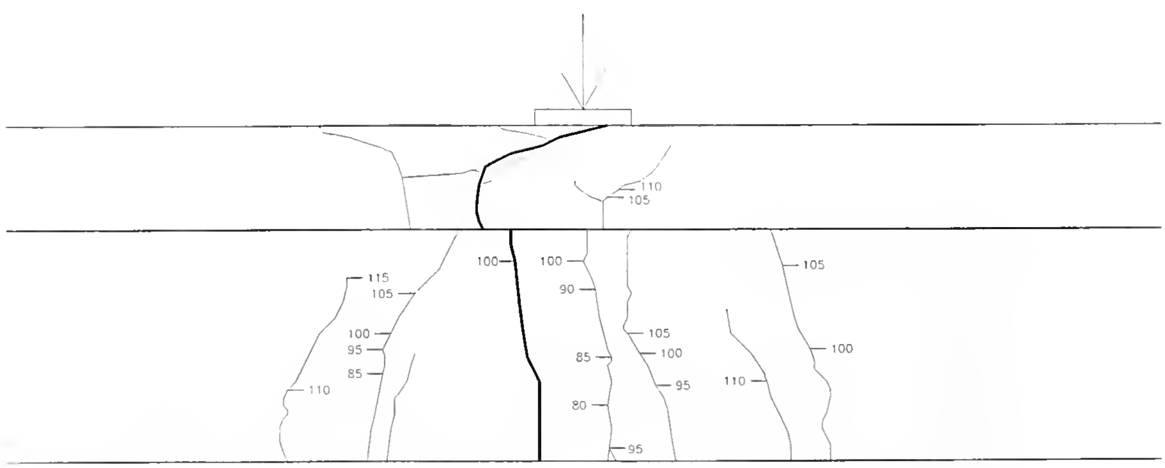
T-Beam FWC-3” Stirrup Spacing (North Face)

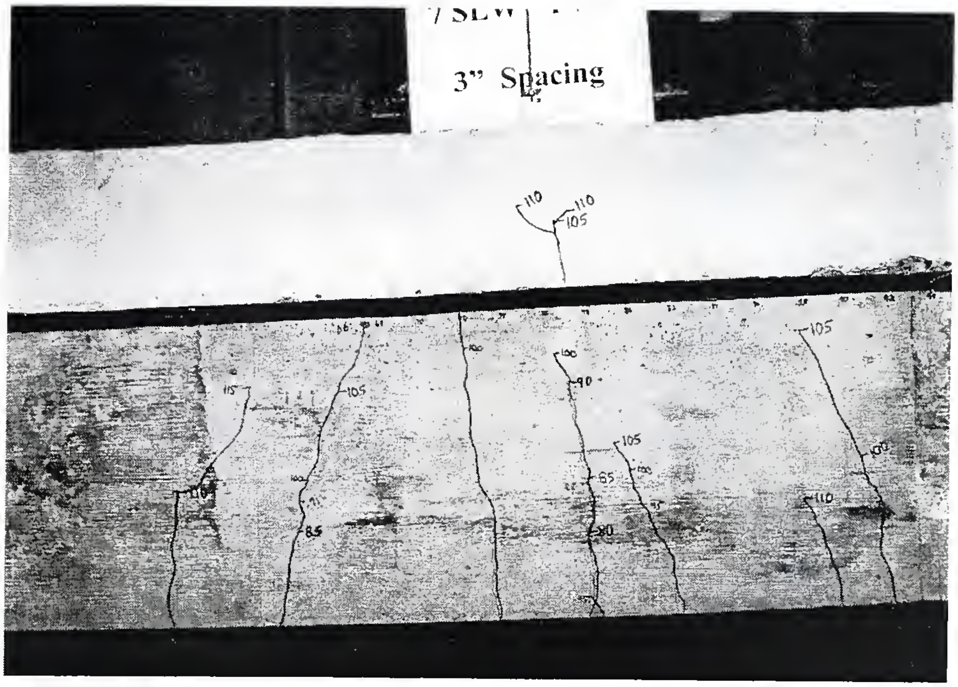
T-Beam FWC-3” Stirrup Spacing (North Face)



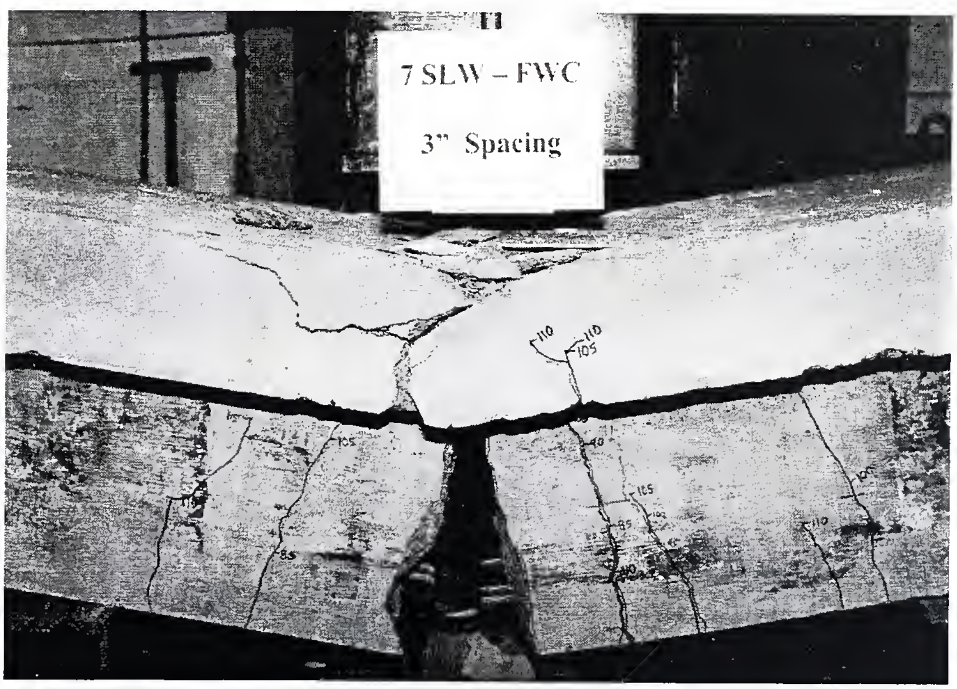
T-Beam FWC - 3" Stirrup Spacing

Crack Patterns on South Face

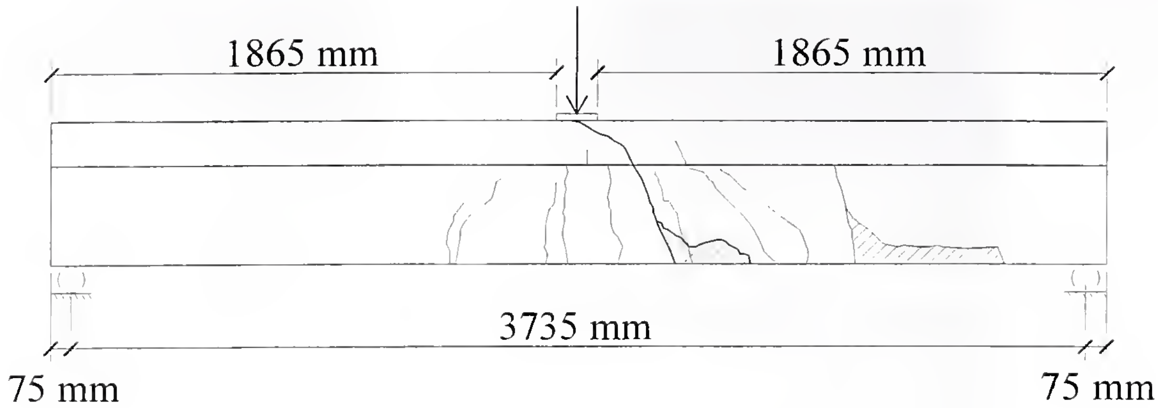




T-Beam FWC-3" Stirrup Spacing (South Face)

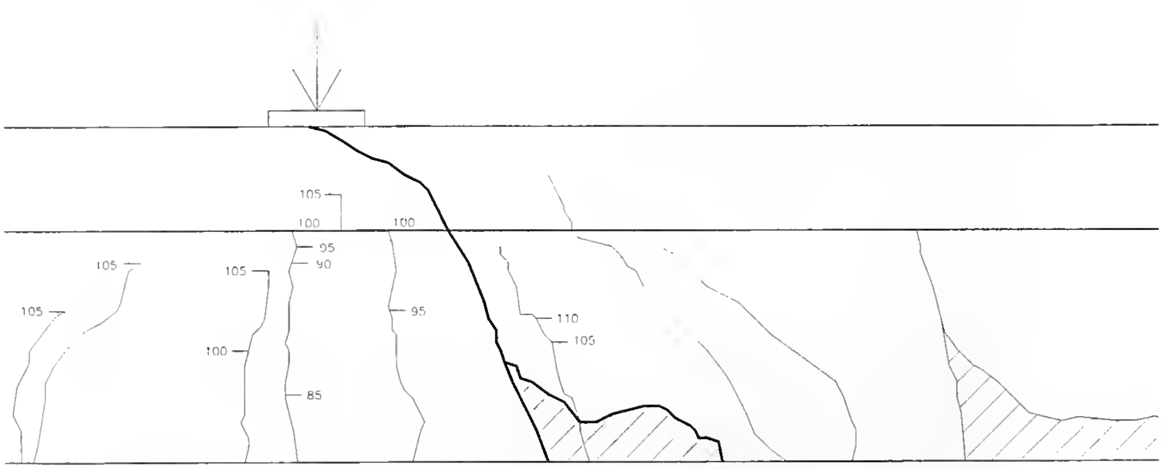


T-Beam FWC-3" Stirrup Spacing (South Face)

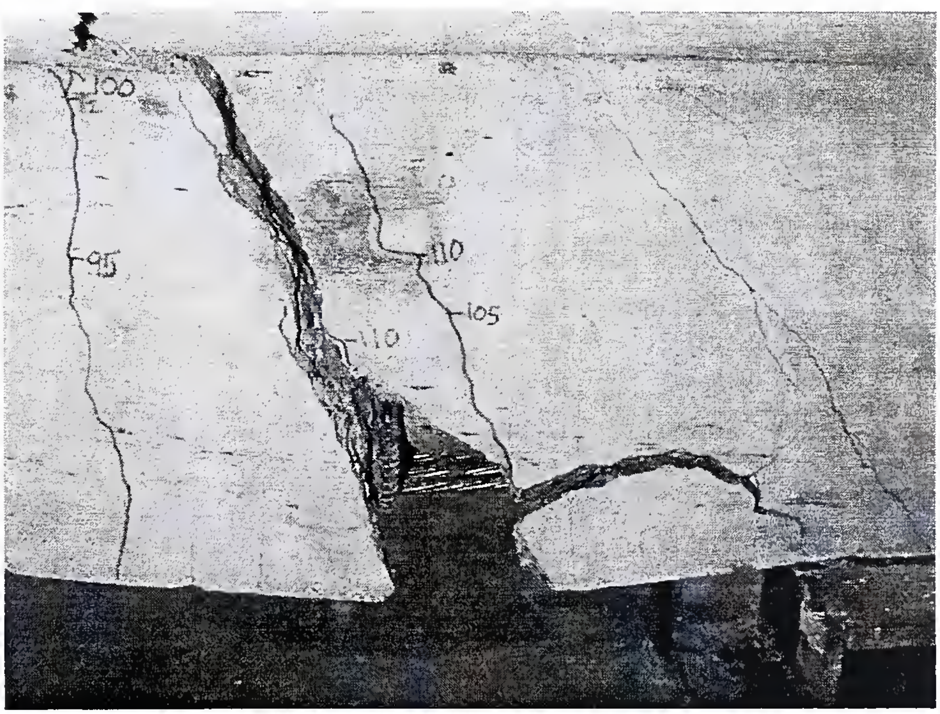


T-Beam FWC - 6" Stirrup Spacing

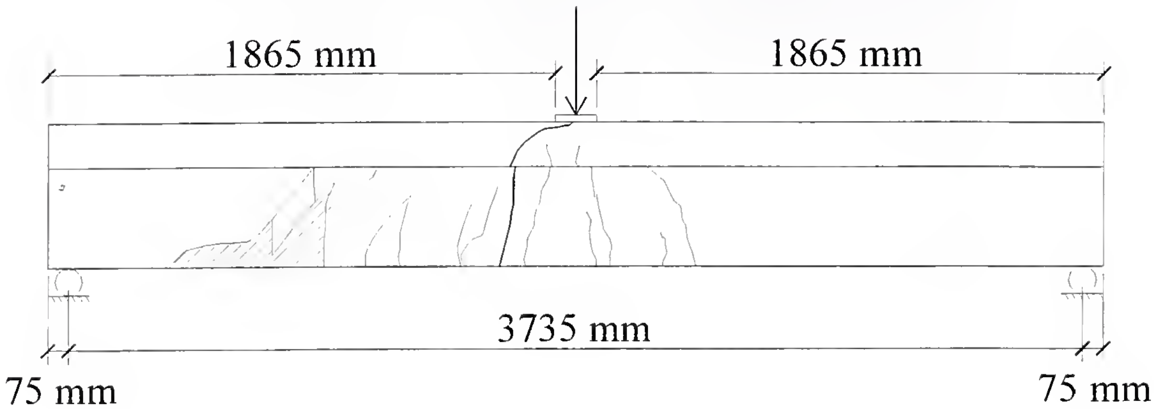
Crack Patterns on North Face



T-Beam FWC-6" Stirrup Spacing (North Face)

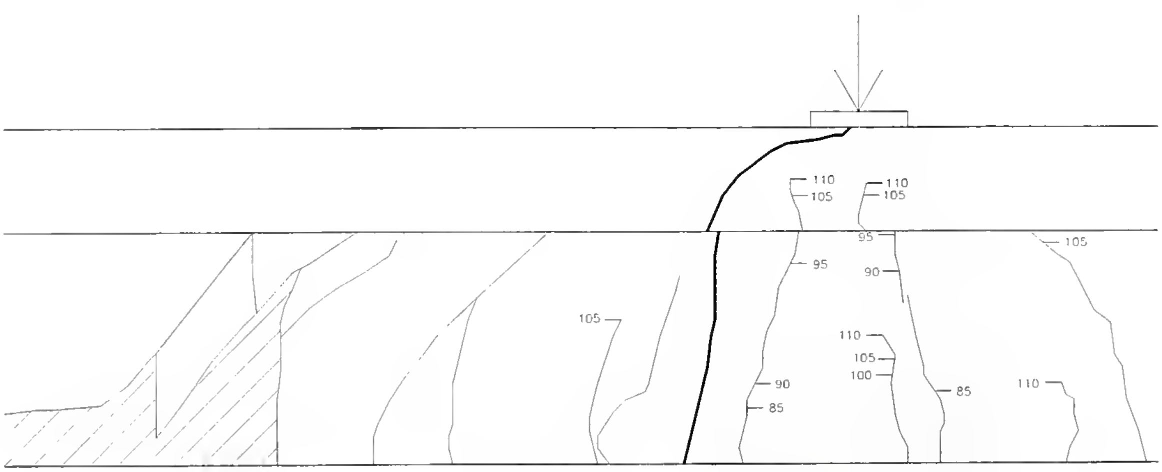


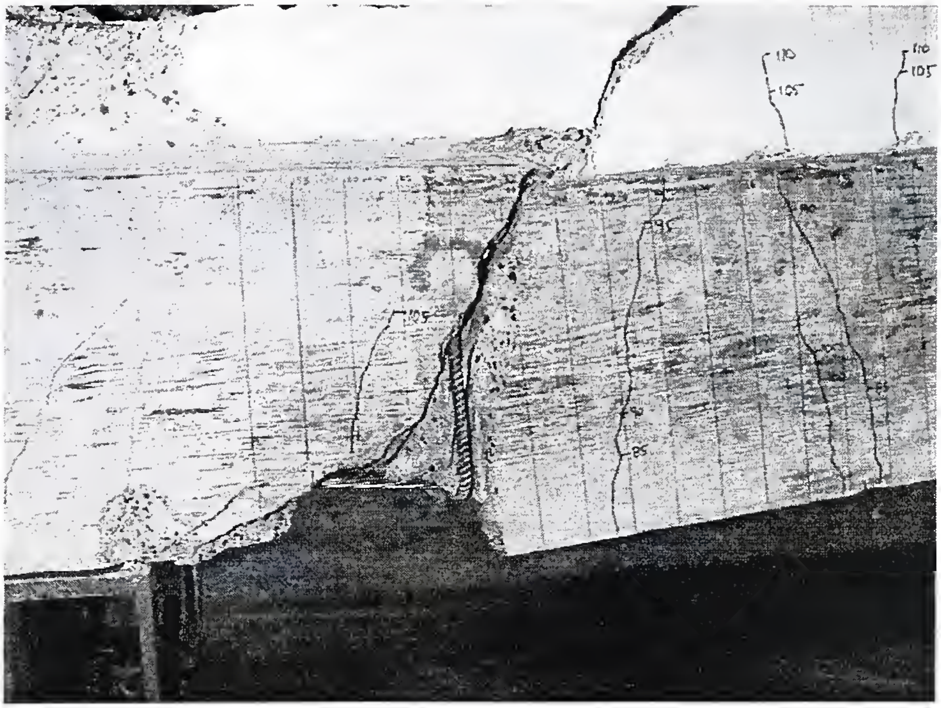
T-Beam FWC-6" Stirrup Spacing (North Face)



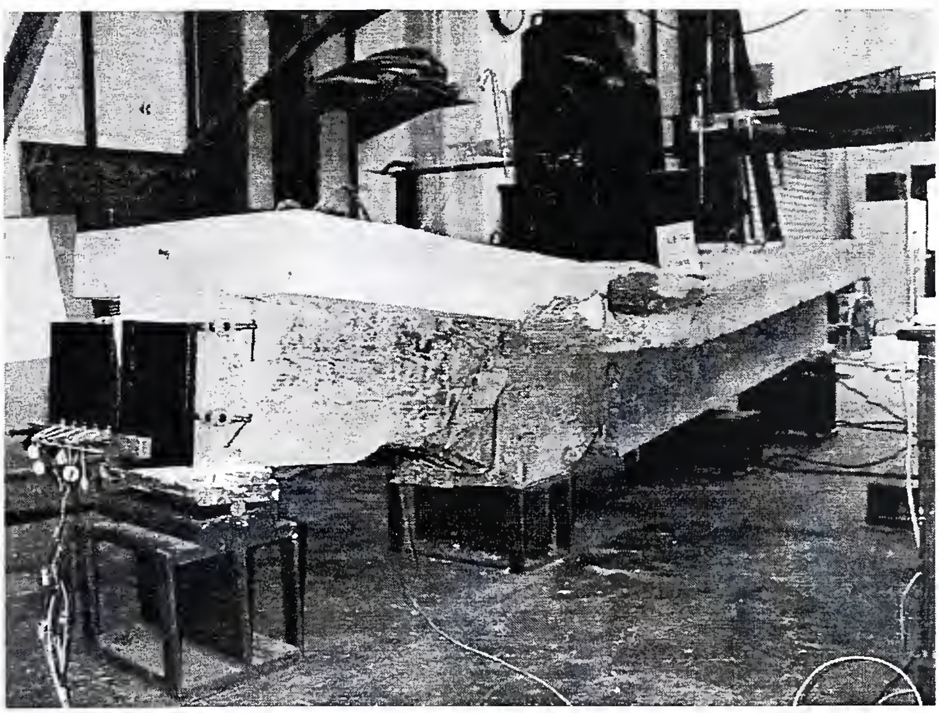
T-Beam FWC - 6” Stirrup Spacing

Crack Patterns on South Face

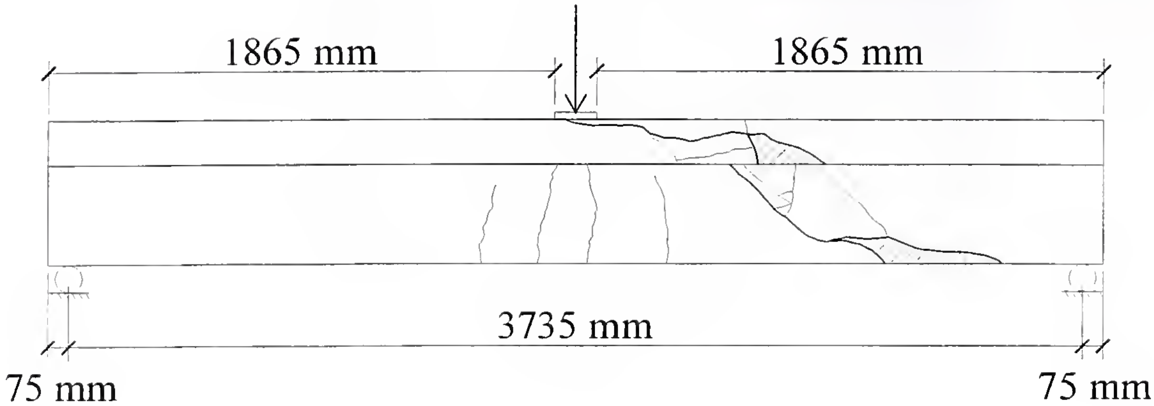




T-Beam FWC-6" Stirrup Spacing (South Face)

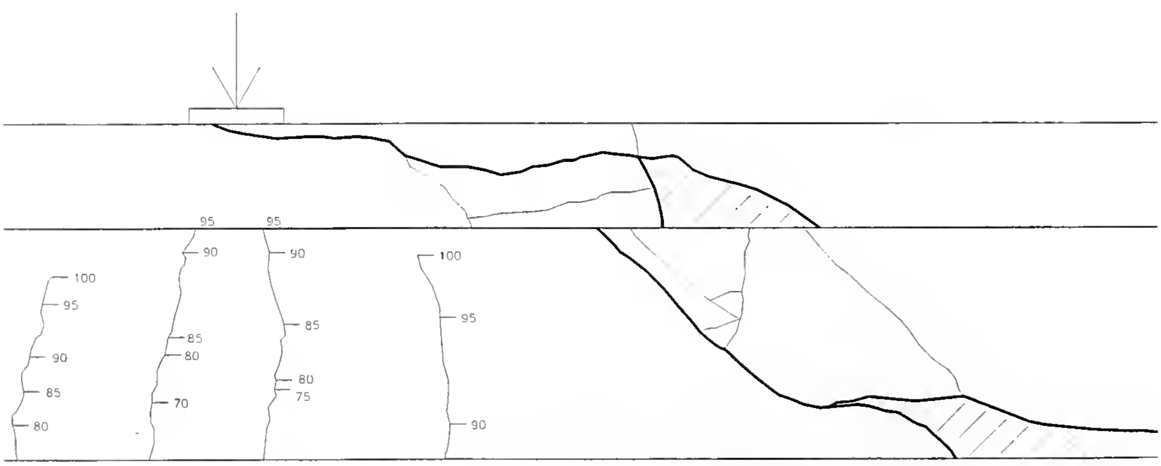


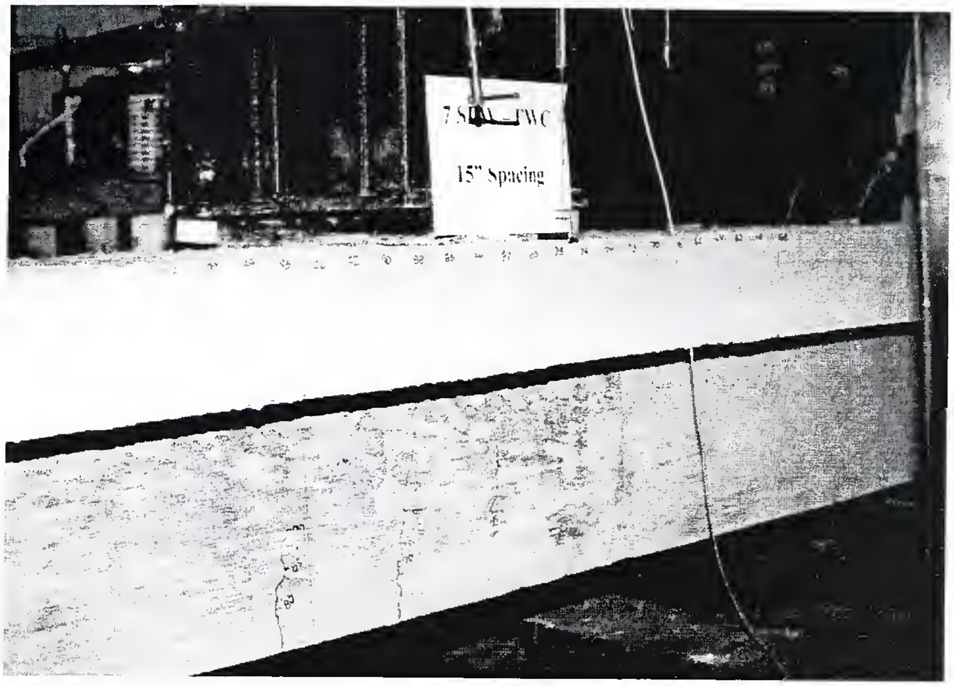
T-Beam FWC-6" Stirrup Spacing (South Face)



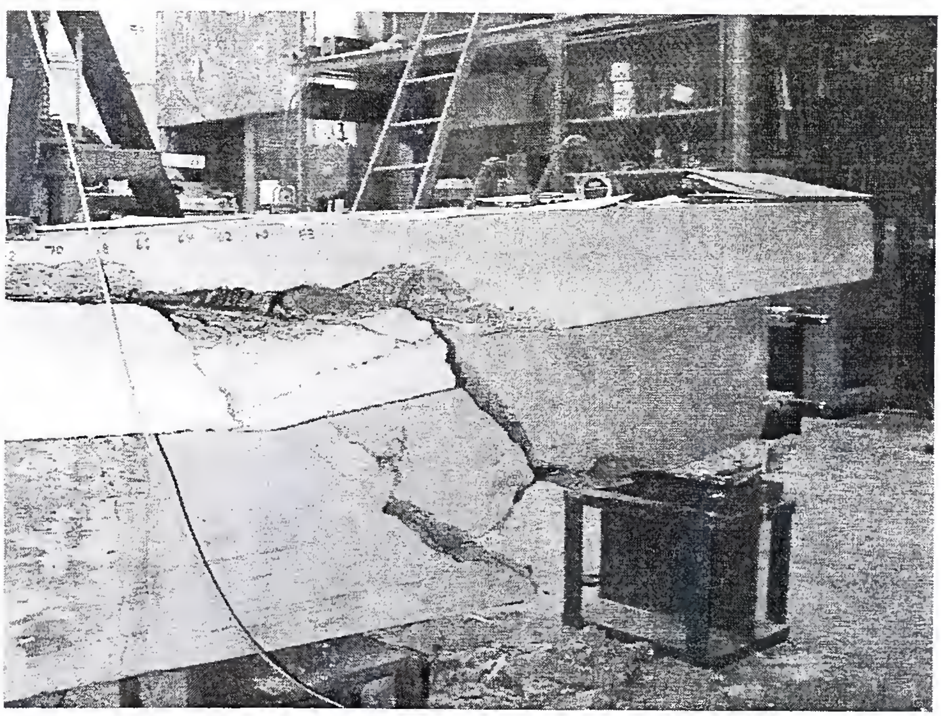
T-Beam FWC - 15" Stirrup Spacing

Crack Patterns on North Face

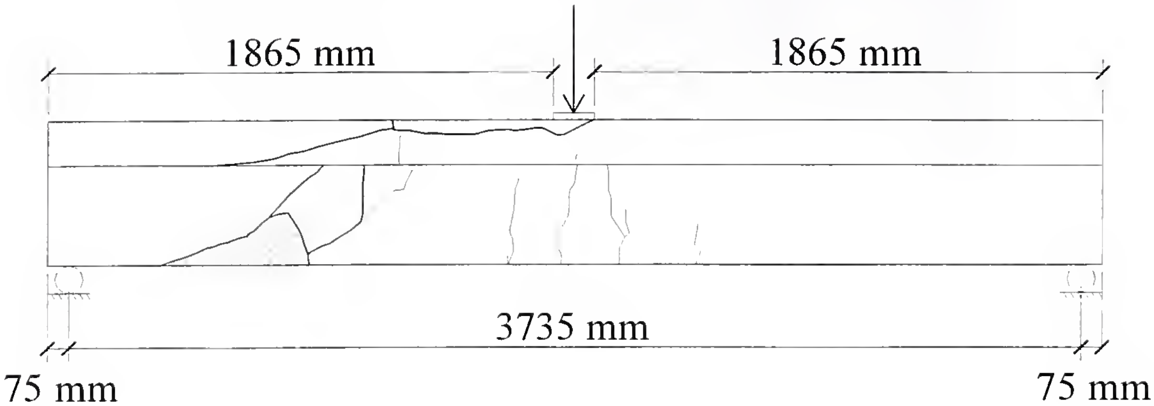




T-Beam FWC-15" Stirrup Spacing (North Face)

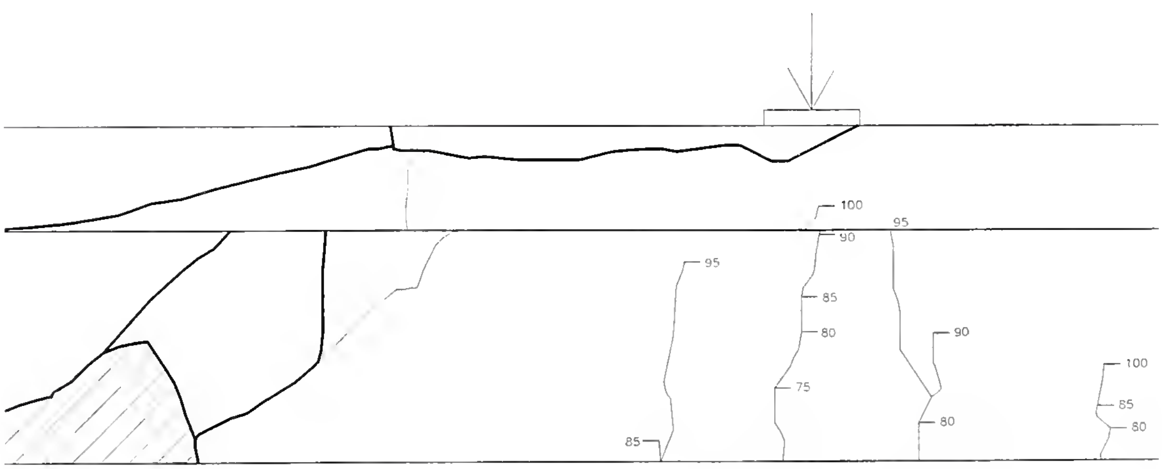


T-Beam FWC-15" Stirrup Spacing (North Face)

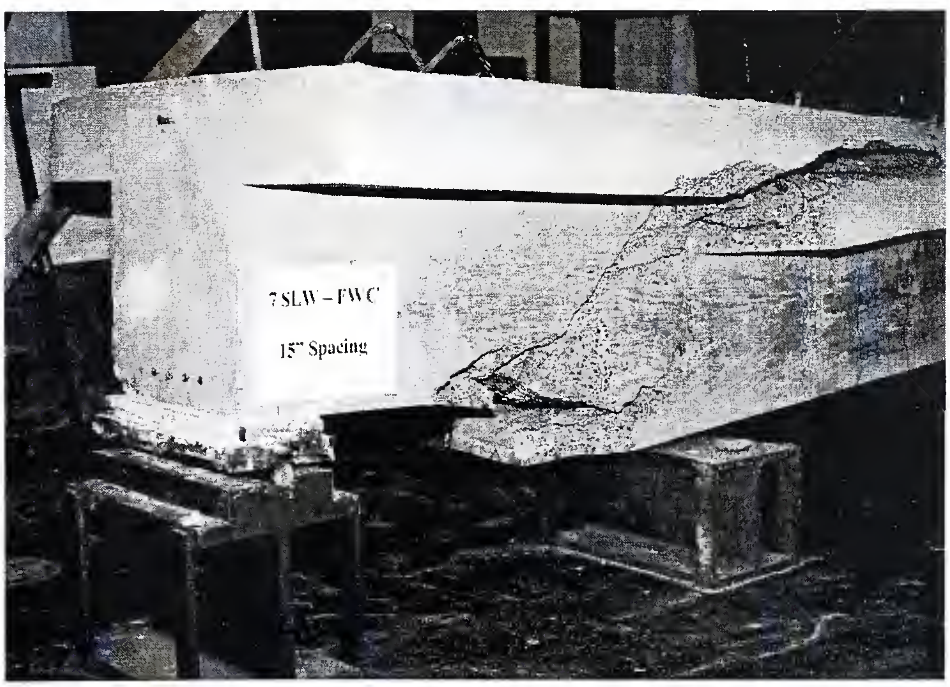


T-Beam FWC - 15" Stirrup Spacing

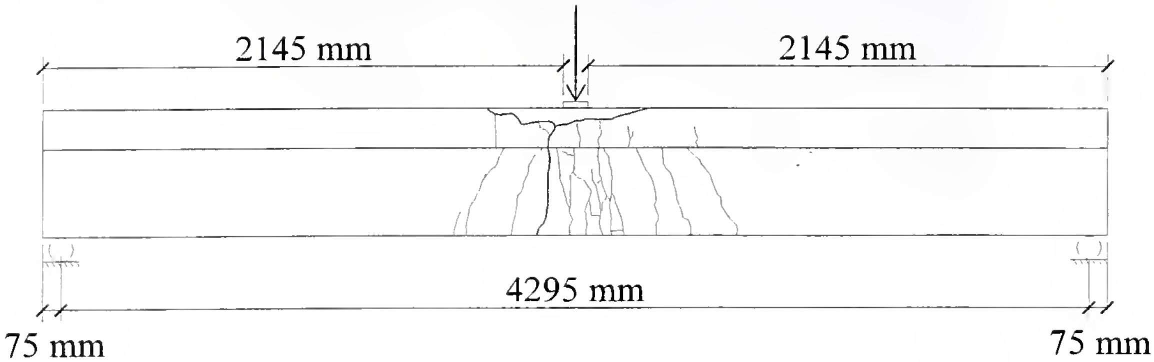
Crack Patterns on South Face



T-Beam FWC-15” Stirrup Spacing (South Face)

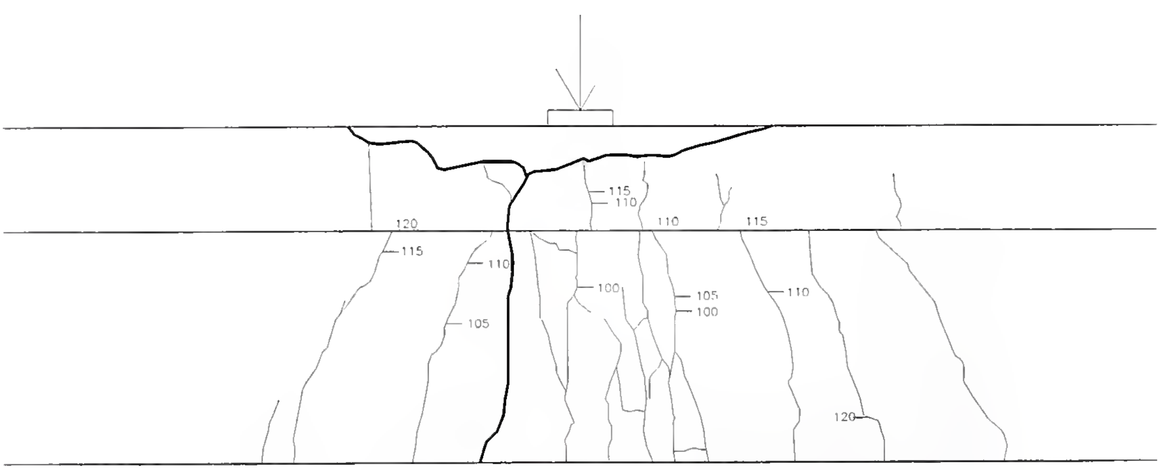


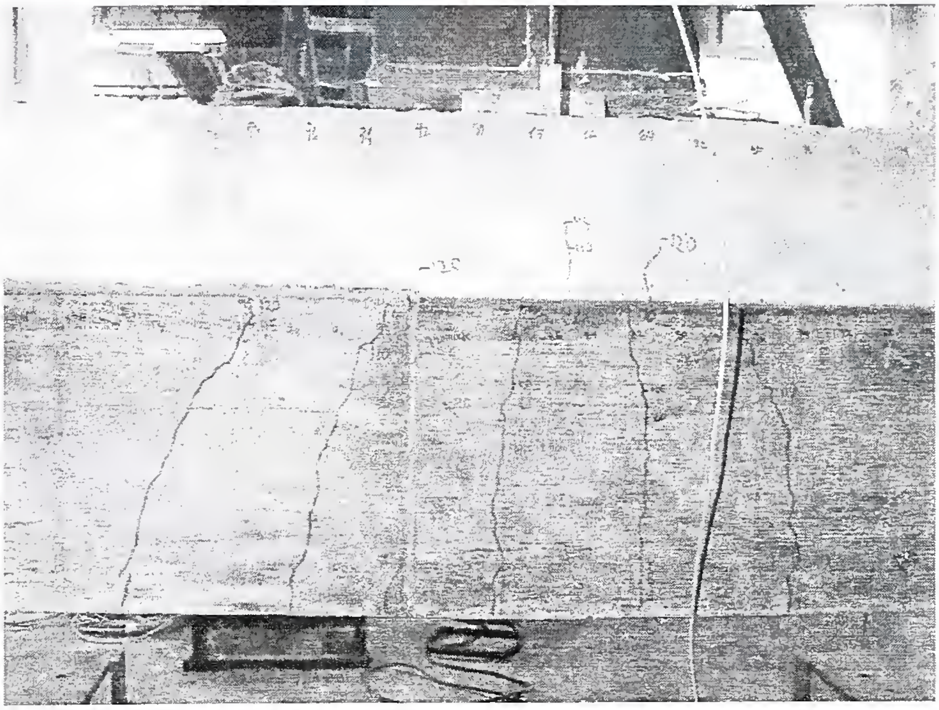
T-Beam FWC-15” Stirrup Spacing (South Face)



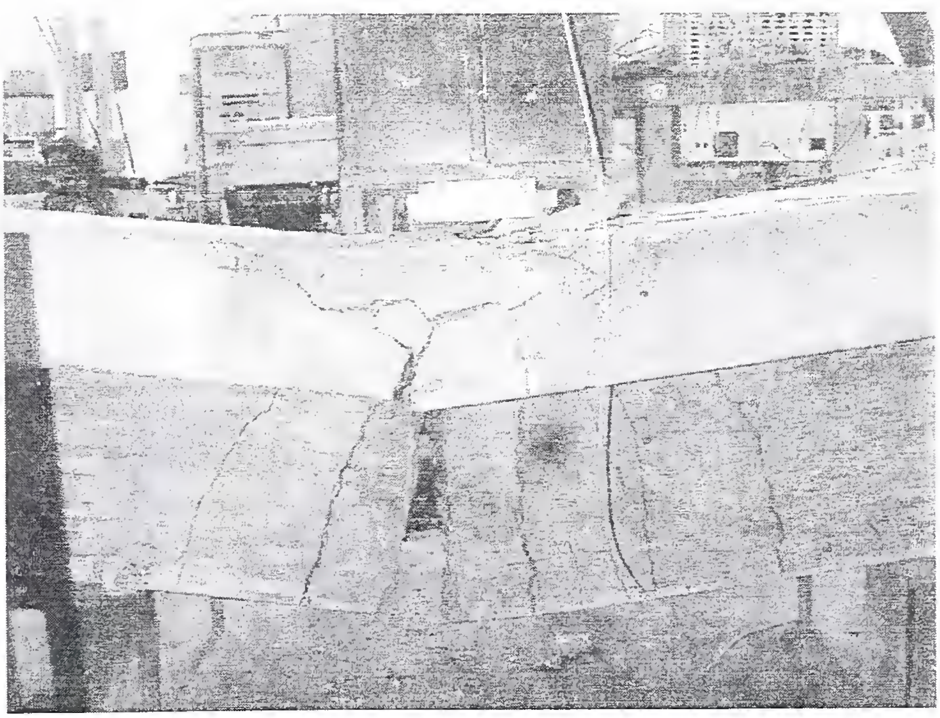
T-Beam 10SLW-0.6

Crack Patterns on North Face





T-Beam 10SLW-0.6 (North Face)



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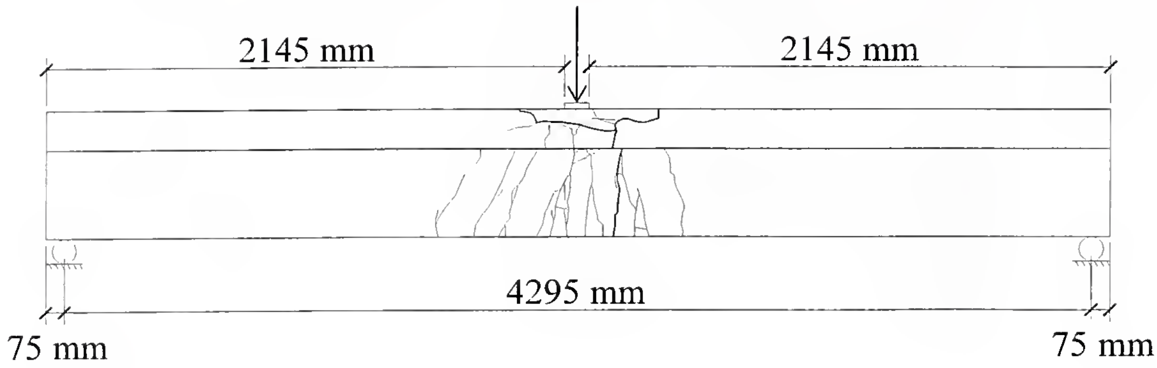
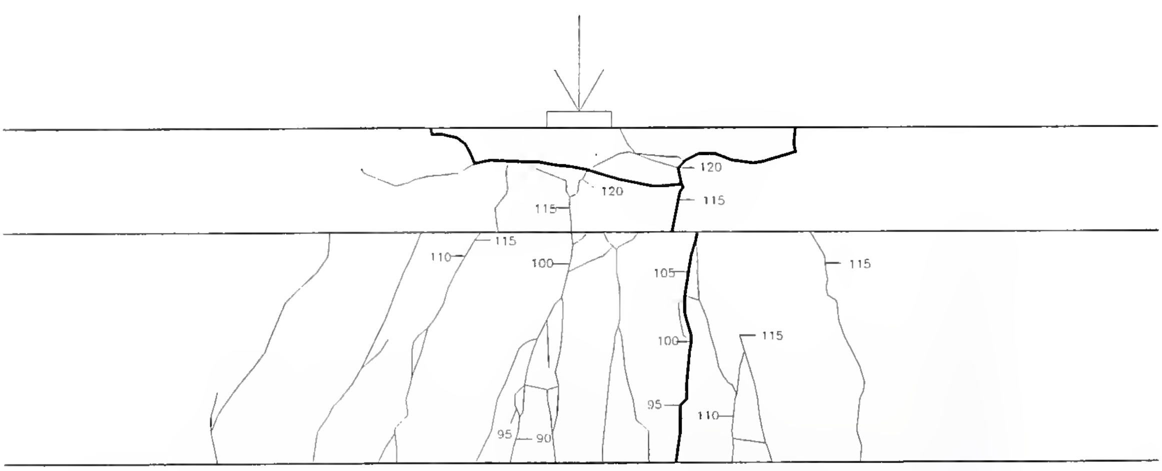
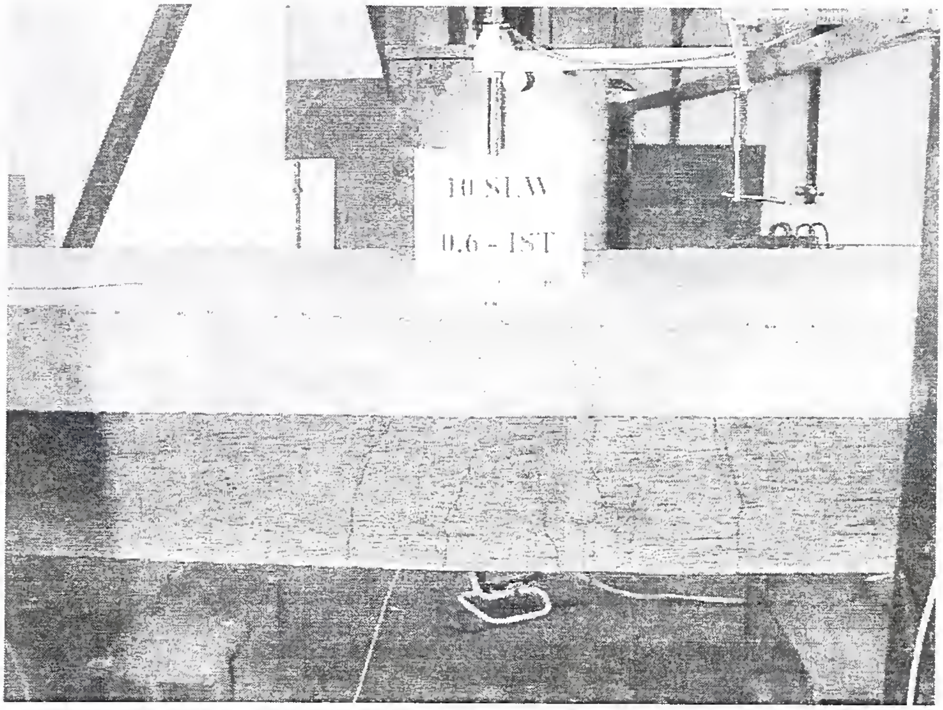


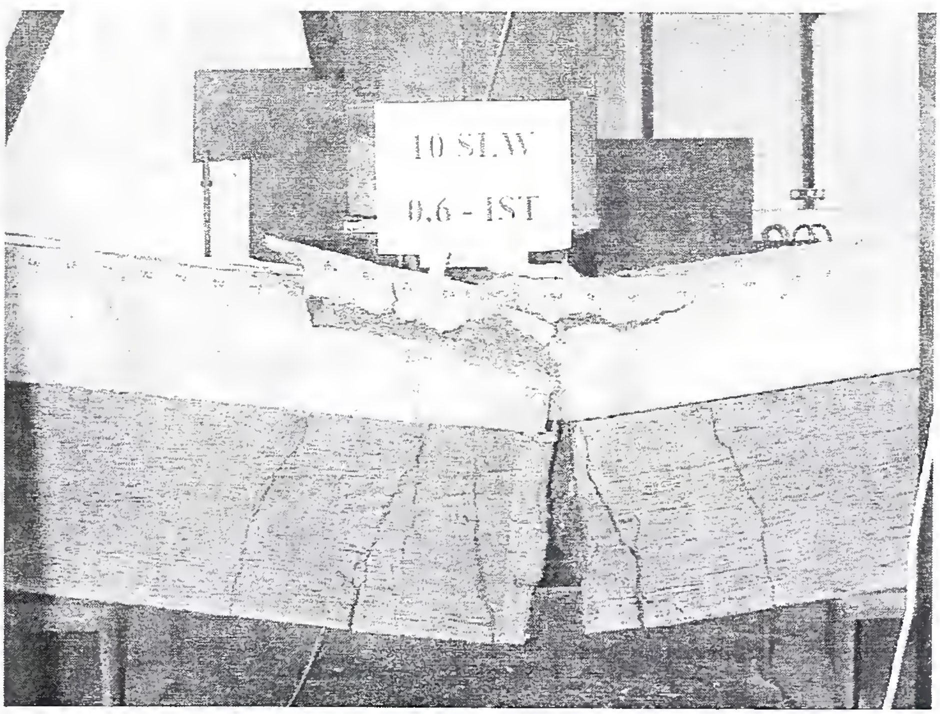
T-Beam 10SLW-0.6

Crack Patterns on South Face





T-Beam 10SLW-0.6 (South Face)



T-Beam 10SLW-0.6 (South Face)

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