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Steven A. Davis - Associate Director

Research Project No. 87-04
Fatigue Testing of Prestressed Beams

SUMMARY REPORT

by

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Office of Research and Sponsored Programs

Bethlehem, Pennsylvania

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ABSTRACT

The research project is discontinued on account of repeated and indefinite postponements of the companion bridge reconstruction project. The report summaries all work completed under the project. Literature survey included the design, inspection and rating computations of the I-95 bridge over Ridley Creek, the effect of skew on the lateral distribution of vehicular live load, and the risk of fatigue failure in prestressed concrete bridge members. Extensive pre-test analyses were carried out on two aspects; the prestress loss in the pre-post-tensioned composite beams in the bridge, and the effect of number and orientation of diaphragms on the distribution of moment resistance. Preliminary observations are drawn from these analysis. Recommendations are provided for needed future research and for modifications to this research project if restarted in the future.

1. <u>INTRODUCTION</u>

1.1 <u>History of Project</u>

The project was started on June 1, 1988. The original work schedule covered a 30 month period and was based on the demolition of the Ridley Creek Bridge in late 1988. The research plan included work on the following five tasks.

    Task 1 - Literature Search
    Task 2 - Field Studies and Evaluation
    Task 3 - Removal, Shipment and Storage of Beams
    Task 4 - Laboratory Testing and Evaluation
    Task 5 - Final Reports

A detailed work plan is included herein as Appendix A. As can be seen, a large portion of the proposed work on this project involved both in-service and laboratory testing of the prestressed concrete beams of the Ridley Creek Bridge. A procedure for estimating the remaining service life with respect to fatigue and a general method for load rating of existing prestressed concrete I-beam bridges were to be developed.

At the end of July 1988, it was learned that the reconstruction of the bridge was delayed and would be rescheduled for the Spring of 1989. A briefing meeting was held in Harrisburg on November 7, 1988, when modifications to the work plan and schedule were discussed. It was proposed that a new task be added to the project in order to utilize the time before conducting field studies. This proposed new task (Task 6 - Pre-Test Analytical Study) included analytical work on both the effect of skew on load distribution and also the behavior of pre-post-tensioned composite beams.

Late in 1988, the researchers learned that the reconstruction work was being further delayed to the Spring of 1990. To accomodate this new delay, the proposed analytical work for Task 6 was further expanded. A formal

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request for an extension to the project and the inclusion of the new task was submitted to PennDOT on March 30, 1989.

In a letter dated May 16, 1989, PennDOT notified the researchers of its decision to teminate all work on the project due to the indefinite delay in the reconstruction of the Ridley Creek Bridge. The researchers were directed to provide a summary report covering all work completed to date.

What follows is a comprehensive summary report of all work completed on Research Project 87-04 between June 1988 and May 1989. Included in this report is information gained from the literature review and pre-test analyses, as well as recommendations for future studies.

1.2 <u>General Description of Bridge</u>

The Ridley Creek Bridge, built in 1963, is a three span twin structure (64'- 96'- 64') carrying Interstate 95 over the Ridley Creek in Chester, PA. Each bridge carries three lanes of traffic on a 40 foot wide roadway that is at a skew of approximately 45 degrees. In each span, the superstructure consists of six pre-post-tensioned concrete I-beams spaced at 8'-0" centers. The roadway surface is a 7-1/2" composite concrete deck with stay-in-place metal forms.

2. LITERATURE SEARCH

2.1 Review Computational and Construction Data for the Ridley Creek Bridge

The following computational and construction data were reviewed to determine the beam design parameters and service history of the structure.

1. Construction Drawings

2. Prestress Beam Shop Drawings

3. Beam Design Calculations

4. 1986 Bridge Rating Calculations

5. 1988 Bridge Inspection Report

The composite pre-post-tensioned beams for the Ridley Creek Bridge were designed in accordance with 1957 AASHO "Standard Specifications for Highway Bridges" (1). All beams were designed as fully prestressed sections capable of carrying HS20-44 live load without any tensile stress in the bottom flange. Each beam was pretensioned with 270 ksi - 7/16 inch diameter seven wire strands and post tensioned with 160 ksi - 1-1/8 inch or 1-3/8 inch diameter special high strength alloy bars. The 28 day design compressive strength of the beam concrete varied between 5250 to 5750 psi. Fascia beams in span 1 and 3 are PennDOT 20/39 I-beams with 32 prestressing strands and one 1-1/8 inch diameter post-tensioning bar. Interior beams in span 1 and 3 are PennDOT 20/39 and 20/36 I-beams respectively. Each beam contains 32 prestressing strands and two 1-1/8 inch diameter post-tensioning bars. In span 2, both the fascia and interior beams are PennDOT 20/54 I-beams with 44 prestressing strand and three 1-3/8 inch diameter post-tensioning bars. Prestress losses were calculated following ACI-ASCE Joint Committee 323 recomendations of 35 ksi for prestressing strands and 25 ksi for post-tensioning bars. The composite concrete deck has a 28 day design strength

of 4000 psi and an effective width of 8'-0". In accordance with the 1957 AASHO Specifications[1], the fascia beams were designed to carry the entire load from the curb and divisor, but a smaller fraction of the live load in comparison with the interior beams. These provisions are no longer contained in the current AASHTO Specifications[2]

Results of the 1986 rating, based on a working stress analysis, indicated that the bridge component controlling the rating was the span 3 interior I-beam. The analysis was performed using the same dimensions and material properties as those in the original design. However, three significant changes were made with respect to other design parameters. Composite dead load of curb and divisor was distributed among all six girders as recommended in current AASHTO specifications. Prestress losses were calculated using the BPR Prestress Loss Equation (3). These losses varied by span from 43.2 to 48.7 ksi for the prestressing strands and from 17.6 to 22.2 ksi for the post-tensioning bars. The use of the BPR equation results in an effective prestress force, and hence, a service load strength approximately 4% lower than values in the original design. Finally, a concrete tensile stress of $3\sqrt{f'_c}$ was allowed under service loads. The BAR5 (Bridge Analysis and Rating-Revision 5) computer analysis indicated an inventory rating of AASHTO H22 or HS28 vehicles for the span 3 interior I-beam. Rating of the exterior girders was not given in the report.

The 1988 Bridge Inspection Report reviewed only the physical condition of the structure and did not include any detailed structural calculations. At the time of the inspection, no distress was noticed in the precast beams. However, areas of concern included a collapsed bearing in span 3, deteriorated and cracked beam pedestals on pier #2 and a severely spalled concrete deck.

2.2 Lateral Load Distribution in Skewed Prestressed Concrete I-Beam Bridges

2.2.1 Lateral Load Distribution in Right Prestressed Concrete I-Beam Bridges

Current AASHTO specifications (2) for load distribution in I-beam bridges is very simplistic and dependant only upon the transverse spacing of the beams:

$$DF = S/5.5$$

where   DF = Fraction of the wheel loads to be carried by each longitudinal beam

         S = Center to center spacing of the longitudinal beams, in feet

It is well known that many other factors affect the lateral load distribution, and hence, the distribution factors. The more important factors include: bridge width-span ratio (aspect ratio), beam spacing to span ratio, flexural and torsional stiffnesses of the entire bridge superstructure, number, location and stiffness of diaphragms or cross bracings, and skew. A series of reports from a previous PennDOT research project contains much detailed information on the effects of these factors and presents proposed refined distribution factor formulas (4).

2.2.2 Effect of Skew

Since the Ridley Creek Bridge has a skew of approximately 45 degrees, special attention was placed on the effect of skew on lateral load distribution in the literature survey. It should be noted that the angle of skew of a bridge superstructure is defined as the acute angle between the support line of the superstructure and the longitudinal axis of the beams (Figure 1). Therefore, a skew angle (Φ) of 90 degrees indicates that the structure is a right bridge (Figure 1a), and a large skew angle reflects a

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bridge with a less severe skew. Although numerous studies had been completed in lateral load distribution before the 1970's, very little work before or since that time have focussed specifically on the effects of skew. Two significant studies were conducted by VanHorn and Kostem at Lehigh University and by Bakht and Moses of the Ontario Ministry of Transportation and Communication and Case Western Reserve University respectively.

The research at Lehigh University was primarily funded by PennDOT and consisted of two separate research projects. The work extended from 1965 to 1980 and covered both spread box and I-beam bridges. Research included field and experimental studies under service load conditions, as well as computerized analysis which extended through the overload stage.

Initially, thirty skewed bridges of various widths, beam spacings, span lengths, skew angles and number of beams were analyzed using the Finite Element Method of analysis (5). Live load distribution factors were computed for the interior and exterior beams for design vechicle loadings. The computation of the load distribution factors is detailed in Reference 5. This initial analytical investigation provided information into the behavior of these bridges and helped determine the effect of each variable on lateral load distribution.

A more comprehensive parametric study of 120 bridges with configurations commonly encountered in practice was completed by DeCastro and Kostem (6). With the results of this analysis, equations were developed which evaluated the reduction in the load distribution factor for interior and exterior beams on skewed bridges. The percent reduction is defined as the amount of reduction required in the distribution factor when a right bridge becomes skewed and therefore is always zero for right bridges. From these studies, the parameters which significantly influenced the percent

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reduction were found to be the bridge width, beam spacing, span length and skew. Regression analyses resulted in the following equations for the percent reduction for interior beams:

$$PCTR_{INT} = (45\ S/L + 2W_c/L)\ cot^2\Phi$$

Where

PCTR$_{INT}$ = Reduction factor to be applied to the distribution factor for an interior beam of a right bridge.
S = Beam Spacing
W$_c$ = Curb-to-Curb Width
L = Span Length
= Skew Angle

For exterior beams, a simplified equation was determined by trial and error:

$$PCTR_{EXT} = 50\ (S/L - 0.12)\ cot\Phi$$

Where

PCTR$_{EXT}$ = Reduction (positive) or increment (negative) to be applied to the distribution factor for an exterior beam of a right bridge.

The above equations are limited to the following bridge dimensions:

$$4'-6" \leq S \leq 9'-0"$$
$$48'-0" \leq L \leq 120'-0"$$
$$30^o \leq \Phi \leq 90^o$$

Although the skewed bridge superstructure is commonly found in practice, very little research effort has gone into this area since the work done by Lehigh University in the 1970's. However, in a recent article, Bakht and Moses have reviewed the practice of analyzing skewed bridges as right bridges and also discussed the important factors affecting load distribution in skewed bridges (7). The method of analysis used was the Grillage Analogy Method. Although different methods of analysis were used, Bakht's results are similar to those of the Lehigh University researchers.

From the results of existing research, the following general observations can be made regarding lateral load distribution in skewed prestressed concrete I-beam bridges.

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1. The distribution factor decreases with decreasing angle of skew.

2. The decrease in the distribution factor is gradual from 90 to 45 degrees but is abrupt from 45 to 30 degrees.

3. As span length increases, the amount of reduction in the distribution factor due to skew decreases.

4. For exterior beams, smaller reductions are obtained in shorter span bridges and increases in the distribution factor occur in longer bridge spans.

5. The bridge aspect ratio, beam spacing-to-span ratio, and skew angle significantly affect the amount of reduction.

2.3 Fatigue of Prestressed Concrete Beams

Fatigue characteristics of prestressed concrete beam members have been the subject of research studies since the beginning of the engineering usage of prestressed concrete (8), and the studies have continued until recent years (9, 10, 11, 12). Most research has focussed on the fatigue behavior of the steel elements, leading to recommended limitations on stress ranges in these elements. Some attention has also been given to the fatigue of concrete, which is primarily in compression. Recommended stress range limitations to insure against fatigue failure have been summarized in reports by ACI Committee 215, Fatigue of Concrete Structures (13) and ACI Committee 343, Reinforced Concrete Bridge Structures (14). The current recommended limitations are as follows:

1. Prestressed strands: $0.10 \, f_{pu}$
   where f_{pu} is the ultimate strength of the strands

2. Non-prestressed deformed bars: $21 - 0.33 \, f_{min} + 8\theta$, in ksi
   where f_{min} is the minimum tensile stress in ksi and θ is
   the ratio of base radius to height of rolled-on transverse
   deformation which, may ordinarily be taken as 0.3

3. Concrete in compression: $0.4 \, f'_c - 0.5 \, f_{cm}$
   where f'_c is 28 day compressive strength of concrete
   and f_{cm} is minimum compressive stress in concrete

The limitation for non-prestressed deformed bars has been incorporated into the AASHTO Bridge Specifications (2) while the others have not.

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It is easy to confirm that fatigue failure rarely occurs in "fully-prestressed" members, in which concrete fiber stress remains always in compression and flexural cracks do not develop. In these members, the stress range in prestressed strands due to live load is extremely low and rarely exceeds a few kips per square inch. These values represent only a small fraction of the fatigue limiting stress range. Thus, earlier research work which dealt primarily with fully-prestressed members generally led to the conclusion that fatigue was not a serious concern. Interestingly, the same conclusion applies to ordinary non-prestressed reinforced concrete members.

The situation was significantly changed with the gain of popularity of partially prestressed concrete members. In this type of members, prestressed strands and non-prestressed bars are combined to provide the required bending moment strength. Tensile fiber stresses are permitted in precompressed concrete. In extreme cases, the specified allowable tensile stress exceeds the modulus of rupture of concrete and the member is actually designed to be cracked under full service load. Naaman has demonstrated that in partially prestressed members, the stress ranges in both prestressed and unprestressed steel are considerably higher than those in comparable fully prestressed or totally unprestressed members (15,16). Consequently, partially prestressed members are more susceptible to fatigue distress than fully prestressed members. Experimental studies at Lehigh University (17), Portland Cement Association Laboratories (18) and University of Texas (12) have demonstrated that the risk of fatigue failure is much more serious for partially prestressed concrete members.

Another factor which could affect the fatigue strength of a structural member is the complexity of its structural details. From studies of the

fatigue behavior of steel structural members, it has been well established that fatigue is controlled by the local stress range which is strongly influenced by the arrangement of the detail. Steel structural details are classified into categories A through F to reflect this influence (2, 19). In a similar manner, the fatigue strength of prestressed concrete members is affected by the structural details. For partially prestressed concrete members, the structural details may include:

1. Rolled-on deformation of unprestressed reinforcing bars

2. Debonding of prestressing strands

3. Deflection of prestressing strands

4. Uneven contact between post-tensioned tendon and its conduit

5. Discontinuity and loss of bond at any transverse crack

A preliminary study at Lehigh University (20) has shown that a flexural crack which does not break the bond would cause a very high stress concentration, and hence a very severe fatigue condition. Very little quantitative information is available regarding the effect of local discontinuity on the fatigue strength of prestressed concrete members.

3. PRE-TEST ANALYSIS

3.1 Introduction

Pre-Test analyses of the superstructure of Ridley Creek Bridge were carried out in order to provide guidance to the instrumentation for the field and laboratory testings. In the original planning, reconstruction work was to start in August of 1988, only two months after the beginning of the project. Only a small amount of analyses, on a very approximate basis, was intended. The postponement of the reconstruction project to spring of 1989 afforded the researchers valuable time to expand this part of work. In all, two aspects were studied: the prestress loss analysis of the pre-post-tensioned beams, and the effect of interior intermediate diaphragms.

3.2 Analysis of Pre-Post-Tensioned Composite Beams

One of the unique features of the Ridley Creek Bridge is the combination of pre- and post-tensioning in the precast concrete beams. The behavior of these beams is further complicated by the cast-in-place concrete deck, with stay-in-place metal deck forms, which acts compositely with the beams. Accurate analysis of structures of this type requires that special attention be given to the time-differential between tensioning of pretensioned strands, casting of the concrete beams, the transfer of pre-tensioning stress, the stressing of post-tensioned tendons and the casting of the deck slab. In addition, the effect of differential shrinkage and the presence of metal deck forms must be taken into consideration. The design provisions contained in the AASHTO as well as PennDOT regulations did not include all these considerations.

An earlier PennDOT research project (project 80-23), conducted by one of the co-principal investigators of the present project had produced a method for the detailed estimation of prestress losses in pre-post-tensioned

concrete beam members (21). As part of the pre-test analysis of the Ridley Creek Bridge, the "Lehigh method" was used for a detailed loss analysis in order to obtain a better estimation of the state of stresses in the bridge beams.

The Lehigh method includes a computerized procedure and a more simplified manual procedure. The manual procedure was used in the pre-test analysis. The prestress loss analysis was performed for an interior beam in span 1, as this was chosen to be the focus of field and laboratory testing (see Section 4). As design and fabrication information gathered from PennDOT was not complete, a number of assumptions had to be made, regarding the fabrication schedule and material properties. This assumed information, as well as information available from prestress beam shop drawings is listed below:

Fabrication schedule: Time from the transfer of pretensioning stress (and the end of curing of beam)

Tensioning of pretensioning strands: -1 day (tensioning at one day before transfer)

Post-tensioning of tendons: 7 days

Casting of deck slab: 180 days

Concrete material properties: Since no fabrication data was available, the concrete strengths were assumed to be identical to those specified.

Beam concrete transfer strength: 4500 psi

Beam concrete compressive strength at post-tensioning: 4800 psi

28 day compressive strength: 5250 psi

Initial modular ratio: 7.3

Long term modular ratio: 7.0

Average loss characteristics: (SRL)

Post tension bars: 36.0 ksi

Prestressing strands: 45.5 ksi

Deck concrete 28 day compressive strength: 4000 psi
                Modular ratio (to beam concrete) 0.87

Pretensioned strands: Seven Wire 270 ksi stress-relieved strands

Post-tensioned tendons: 160 ksi Stressteel Bars

Jacking stress: 0.73 f's = 116.9 ksi

Friction coefficient: 0.15

Wobble coefficient: 0.0002/foot

Anchorage seating distance: 0 in.

Based on these values, the prestress loss of an interior beam in span 1 was calculated. Since the bridge was built in 1963, losses were calculated for an age of 25 years (1988). The results were:

For pretensioned strands: 67.3 ksi, or 35.6%
For post-tensioned tendons: 42.3 ksi, or 36.2%

In contrast, the current AASHTO Specifications (2) provide prestress loss values as follows:

For pretensioned strands: 53.1 ksi, or 28.1%
For post-tensioned tendons: 37.8 ksi or 33.8%

It is seen that the more detailed Lehigh method predicted significantly higher losses than the provisions of the AASHTO Specifications. These losses are also greater than values used in the original design as well as the recent rating computations (Section 2.1). An experimental determination of the remaining prestress was included in Task 4 of this project. Without such an experimental determination, it is not possible to evaluate the different estimations.

As pointed out earlier (Section 2.1), the design of this bridge was based on the 1957 AASHO (ASCE-ACI) provisions for prestress losses and allowable stresses and a live load of the HS20-44 class. Using the prestress losses predicted by the Lehigh method would increase the

calculated beam stresses. In particular, the computed lower fiber concrete stress, under HS20-44 live load and impact, would be 658 psi in tension, which would be approximately 3 times the allowable stress ($3 \sqrt{f'_c}$) and 21% over the predicted modulus of rupture ($7.5 \sqrt{f'_c}$).

Since the "modulus of rupture" of concrete signifies the development of flexural cracks in a concrete beam member, a tensile stress exceeding this limit is theoretically impossible to occur. The calculated high stress indicates only that the section is expected to be cracked. However, cursory preliminary inspection by the researchers (and PennDOT routine inspections in 1986 and 1988) has revealed no visible transverse cracks in the beam. The contradicting evidence may be explained by one or more of the following:

1. The dead and live load stresses were computed based on AASHTO specified distribution factors. These factors were conservatively developed for the purpose of design, and were based on very much simplified models of the superstructure. The actual structure is much more complex, particularly in view of the severe skew and the stay-in-place metal deck. The actual dead and live load stresses are conceivably much lower than the calculated values.

2. The computation was based on the specified concrete strengths. In reality, the strengths were probably higher.

3. The researchers have not been able to acquire any record of the actual traffic on this bridge. Whether the live load (and impact) has been as high as represented by the HS20-44 standard is open to question.

3.3 Effect of Interior Diaphragms on Load Distribution in Skewed Prestressed Concrete I-Beam Bridges

3.3.1 Background

Substantial effort was made in an investigation into the effects of interior diaphragms on load distribution in skewed prestressed concrete I-beam bridges. A Finite Element analysis parametric study was conducted on south bound span 1 of the Ridley Creek Bridge which carries three lanes of traffic on a 40 foot wide roadway that is at a skew of approximately 45 degrees. The superstructure consists of six PennDOT 20/39 pre-post-tensioned concrete I-beams, topped with a 7-1/2" inch composite concrete deck.

3.3.2 Objective

Two parameters were examined for their effects on the load distribution characteristics of a severely skewed (45 degrees) I-beam superstructure.

1. Number and Location of Intermediate Interior Diaphragms

    Four cases were studied: No intermediate diaphragms, one diaphragm at midspan, two diaphragms at third points and three diaphragms at quarter points.

2. Diaphragm Orientation

    Two cases were studied: diaphragms placed perpendicular to the longitudinal axis of the beams and diaphragms placed parallel to the abutments.

As a basis for comparison, the same parameter variations were also used in the analysis of a right (90° skew) bridge of the same span length. Midspan deflections and the midspan moment distribution were used as the principle indicators of the load distribution characteristics.

3.3.3 Finite Element Model

The cross section of the bridge superstructure, not including diaphragms, was idealized into an assemblage of 12 isotropic plate elements at mid-depth of the concrete deck and 6 eccentrically connected beam

elements located at the neutral axis of the composite prestressed concrete
I-beams. Each prestressed beam element was connected to the deck by a very
stiff vertical beam element, 25-3/4" in length, to simulate the full
composite action in the structure. The curb and divisor were modelled by
beam elements located along the outside edges of the overhanging deck.
Since the centroid of the curb and divisor are above the centroid of the
deck, the flexural inertia was modified by $I = I_o + Ad^2$. Longitudinally,
the 64' span was divided into ten 4'-0" segments symmetrical about midspan
and two 6'-0" segments at each end. The idealized bridge cross section and
longitudinal divisions are presented in Figure 2.

Interior diaphragms, 2'-1" high by 0'-9" wide, were modelled by plane
stress elements. Each diaphragm consisted of two elements 25-3/4" deep
attached to the deck and prestressed concrete beam nodes. To account for
the extra depth, the thickness of the plane stress element was modified to
provide the same moment of inertia as in the actual diaphragm.

Diaphragms at the abutment and pier were modelled by beam elements
located along the deck nodes at the end of the structure. The flexural
inertia was modified to account for the eccentricity of the member's
centroid.

3.3.4 Loads

The design live load used for this study is the AASHTO HS20-44 vehicle.
Four load combinations were considered with load cases 1-2 and load cases 3-
4 intended to produce maximum effects in the exterior beam (Beam #6) and
interior beam (Beam #4), respectively. Figure 2 illustrates the lateral
position of the vehicle on the structure for all four load cases.

Longitudinally, the load was placed with the drive wheels of the truck
directly over the midspan line for the right bridge. For the skewed

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structure, the center-of-gravity of the drive axle was placed on the midspan line. These locations will not produce the absolute maximum moment on the structure. However, the deficiency was judged to be small and would not justify the necessary increase in mesh complexity required to determine the absolute maximum moment.

3.3.5 Analysis

The analysis results comparing midspan deflections and midspan moment distribution are presented below.

Moment Distribution

Figures 3-10 illustrate the distribution of moment among the six prestressed concrete I-beams at midspan for the three bridge configurations and various load cases. The percentage of total bending moment carried by a beam was calculated by dividing the moment in an individual beam by the summation of moments in all six beams. The moment in any beam was defined as the sum of two components:

1. Average of the bending moments in the two beam elements adjacent to midspan.

2. The moment of the force couple between the axial loads in the concrete deck and the prestressed concrete beam.

This procedure ignored the bending moment capacity of the concrete deck. However, this was justified since calculations showed that the deck contributed less than 1% of the moment at midspan.

In general, it appears that the moment is more uniformly distributed among the beams when diaphragms are present. The addition of diaphragms appears most benificial in distributing eccentric and localized load conditions as in load cases 1 and 3. For bridges with a perpendicular diaphragm configuration, Figures 3-6, the distribution characteristics improve as the number of diaphragms increase. The maximum percentage of

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total moment carried by an individual beam, or beam moment, in a structure with 3 diaphragms is 12-19% below corresponding values in a structure without diaphragms.

Figures 7 and 8 illustrate the distribution of moment in skewed bridges with diaphragms placed parallel to the abutment for load cases 1 and 3. With one diaphragm at midspan, the reductions in beam moment are 27% and 30% for load cases 1 and 3 respectively. These values are significantly greater than similar values for a structure with as many as three diaphragms placed perpendicular to the beams. A similar result is present in Figures 9 and 10 for right bridges where a 35% reduction is possible with only one diaphragm at midspan.

Earlier work by DeCastro and Kostem at Lehigh University (6) also indicated that only one diaphragm at midspan is most benificial in distributing load on right bridges. In fact, the 35% reduction in maximum beam moment was the largest reduction recorded in this study. This indicates that the skew angle is an important variable in defining the effect of intermediate interior diaphragms on load distribution.

Midspan Deflections

Figures 11-14 and 15-16 illustrate the midspan deflection profile for a skewed bridge with diaphragms placed perpendicular to the beam axis and a right bridge respectively. The plot indicates deflections of the six prestressed concrete beams.

Reductions in midspan deflection due to the addition of interior diaphragms perpendicular to the beams vary between 16-22% depending on the load condition. For diaphragms placed parallel to the abutments, the reduction is 1-4% less than those stated for perpendicular diaphragms. In all three bridge configurations, the number of interior diaphragms had a

direct impact on the maximum beam deflection. In general, as the number of interior diaphragms increase the midspan deflections will decrease.

It can also be seen that the addition of interior diaphragms has a larger impact on deflections in non-skewed structures. However, the deflections in a skewed bridge, regardless of diaphragm orientation, are 15-25% lower than those of a right bridge with similar span length.

3.3.6 Preliminary Conclusions

General conclusions cannot be drawn from this study because of its limited scope. No variation was made in the width of the bridge, length of the span, angle of skew, number and spacing of the longitudinal beams, size of the precast beams and thickness of the slab. Several, if not all, of these foregoing parameters are likely to have an effect on the lateral load distribution characteristics. Nevertheless, based on this limited study, the following preliminary observations can be made.

1. Intermediate interior diaphragms decrease the midspan deflection in the beam directly below the live load.

2. The diaphragm orientation has virtually no effect on midspan deflections.

3. Intermediate interior diaphragms effectively create a more uniform distribution of the midspan bending moments among the longitudinal beams.

4. One diaphragm placed at mispan is most effective in distributing midspan moment in right bridges and skewed bridges with diaphragms placed parallel to the abutments.

5. Increasing the number of diaphragms in a skewed bridge with diaphragms placed perpendicular to the beams gradually improves the load distribution characteristics of the structure.

6. Intermediate interior diaphragms become more effective in distributing midspan moment as the angle of skew is increased.

4. FIELD TESTING AND BEAM REMOVAL

Only a small amount of work was accomplished on Task 2, field testing and Task 3, beam removal. Because of the delay in reconstruction of the Ridley Creek Bridge, the actual field testing and beam removal could not be completed. The only work accomplished in this area was preliminary planning.

Preliminary planning for the testing was initiated by making a visit and cursory inspection of the bridge site with PennDOT District 6-0 personnel in August of 1988. From the ground, no cracks were visible in the precast beams. However, indications of structural distress were noticed in other areas of the bridge. These deficiencies were previously described in the 1988 Bridge Inspection Report (Section 2.1).

Originally, the focus of study was concentrated on span 3 of the north bound structure because of its low rating. However, after discussion with PennDOT District 6-0 personnel and consideration of the reconstruction sequence and access limitations at the site, this original plan was abandoned. Instead, the study became focused on the first span of the south bound structure.

The proposed initial stages of the reconstruction schedule were as follows.

Stage 1: Widen the north bound bridge to 4 lanes.

Stage 2: Close traffic to the innermost south bound lane and the two inner north bound lanes to allow reconstruction of the north abutment (span 3).

Stage 3: Close traffic to the south bound structure to allow its reconstruction.

It was the intention of the researchers to develop a field testing plan which was coordinated with this reconstruction schedule and would therefore

minimize interruption of the contractors activities. Preliminary plans were developed as follows.

1. During the latter part of stage 1, before any modification to the southbound structure, a thorough field inspection will be made (Task 2a) and regular traffic strain measurements will be taken (Task 2b).

2. No field work will be conducted during stage 2.

3. At the beginning of stage 3, when public traffic will be diverted from the south-bound structure, test truck strain measurements will be taken (Task 2c).

4. Immediately after completion of Task 2c, the contractor will begin demolition of the south bound structure. At that time, the removal of prestressed beams (Task 3) for laboratory testing will be completed.

A draft set of detailed specifications for field testing and cutting and removal of prestressed beams was developed and transmitted to PennDOT District 6-0 in August 1988. The complete specification is included in Appendix B.

5. CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE STUDIES

Because of the unanticipated early termination of this project, the various components of this study did not progress sufficiently to generate substantive conclusions. Nevertheless, preliminary conclusions are derived based on the literature review and the pre-test analyses. Also, areas for potential future research are identified and suggestions are offered for consideration if a project similar to this one is to be undertaken in the future.

1. Fatigue is a serious possible failure mode for partially prestressed concrete members. Here "partial prestress" is defined to refer to designs which allow tensile stress in precompressed concrete under service condition, and/or which combines prestressed and unprestressed reinforcement. Extensive research is needed to quantify the fatigue characteristics of these members, particularly with reference to the transverse cracking of concrete and local debonding and change in prestress by blanketing and/or deflecting.

2. The skew of a bridge superstructure has a pronounced effect on the internal moment and shears if the skewness is severe. The current design procedure results in conservative structural design. However, additional research is needed to realistically estimate the internal moments and shears in each beam member for evaluation of fatigue, and to achieve improved economy in the design of severely skewed bridges.

3. The number and orientation of intermediate diaphragms affect the structural behavior of beam-slab type bridge superstructure having a 45° skew angle. More extensive analyses are necessary to fully understand this effect and to generate design guidelines.

4. The combination of pretensioned strands and post-tensioned tendons in the same structural member creates complexities in the analysis process with regard to estimation of prestress losses, behavior under service loads and the ultimate member strength. Reasonably accurate procedures are currently available for these analyses, but simplifications are needed for practical design usage.

5. The Ridley Creek Bridge is severely skewed and also contained pre-post-tensioned beams with a smaller-than-usual depth to span ratio. The several unusual features of this bridge structure will obviously influence its behavior under service, as well as fatigue strength. The intended study would have provided an opportunity to explore the overlapping effects of these features on the behavior and strength of the bridge and the beams. With the termination of the project such an opportunity is lost. It is suggested that field and

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laboratory studies be undertaken in the future, preferably on test structures each with an unusual characteristic so that its specific influence can be thoroughly investigated.
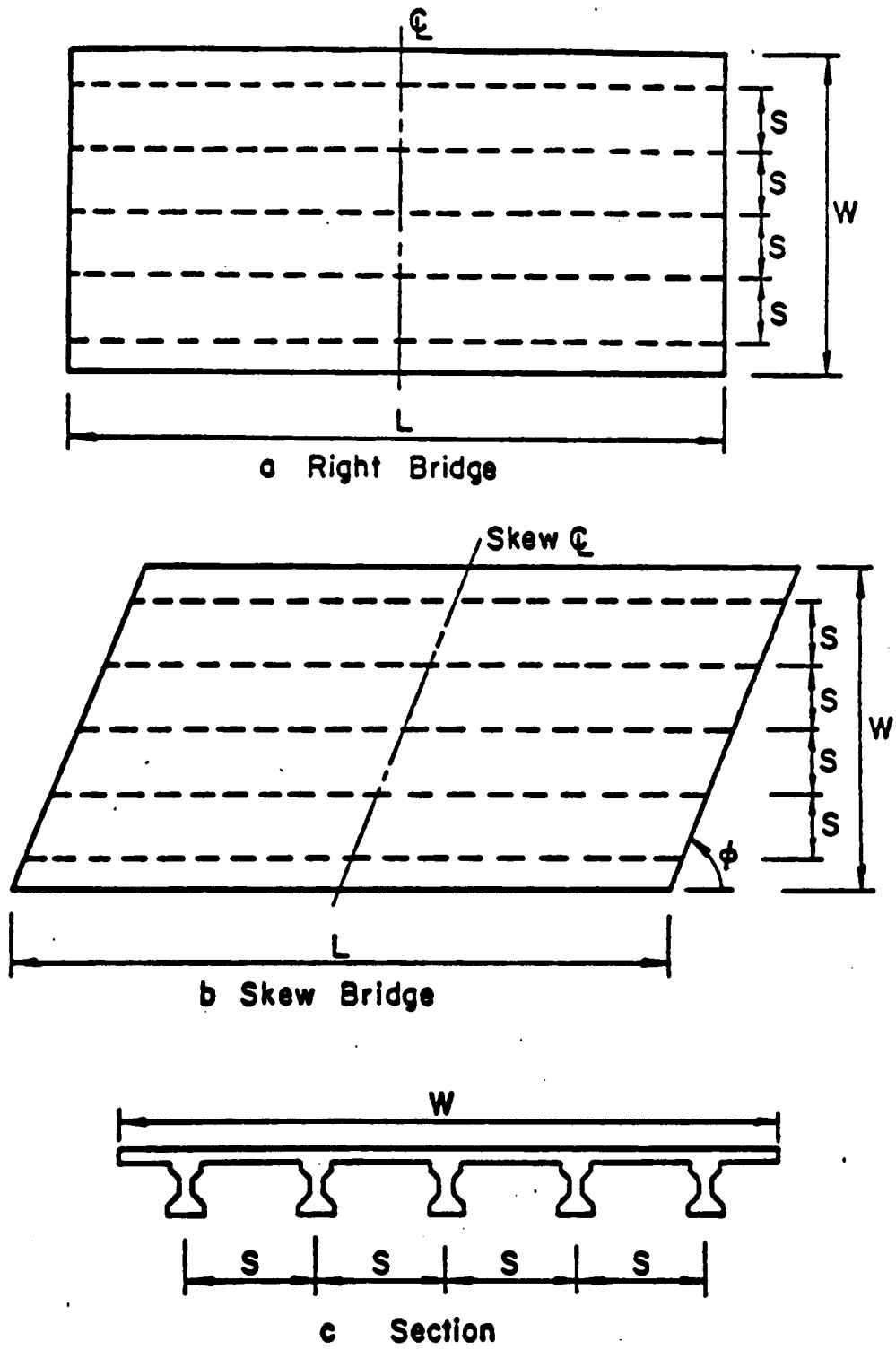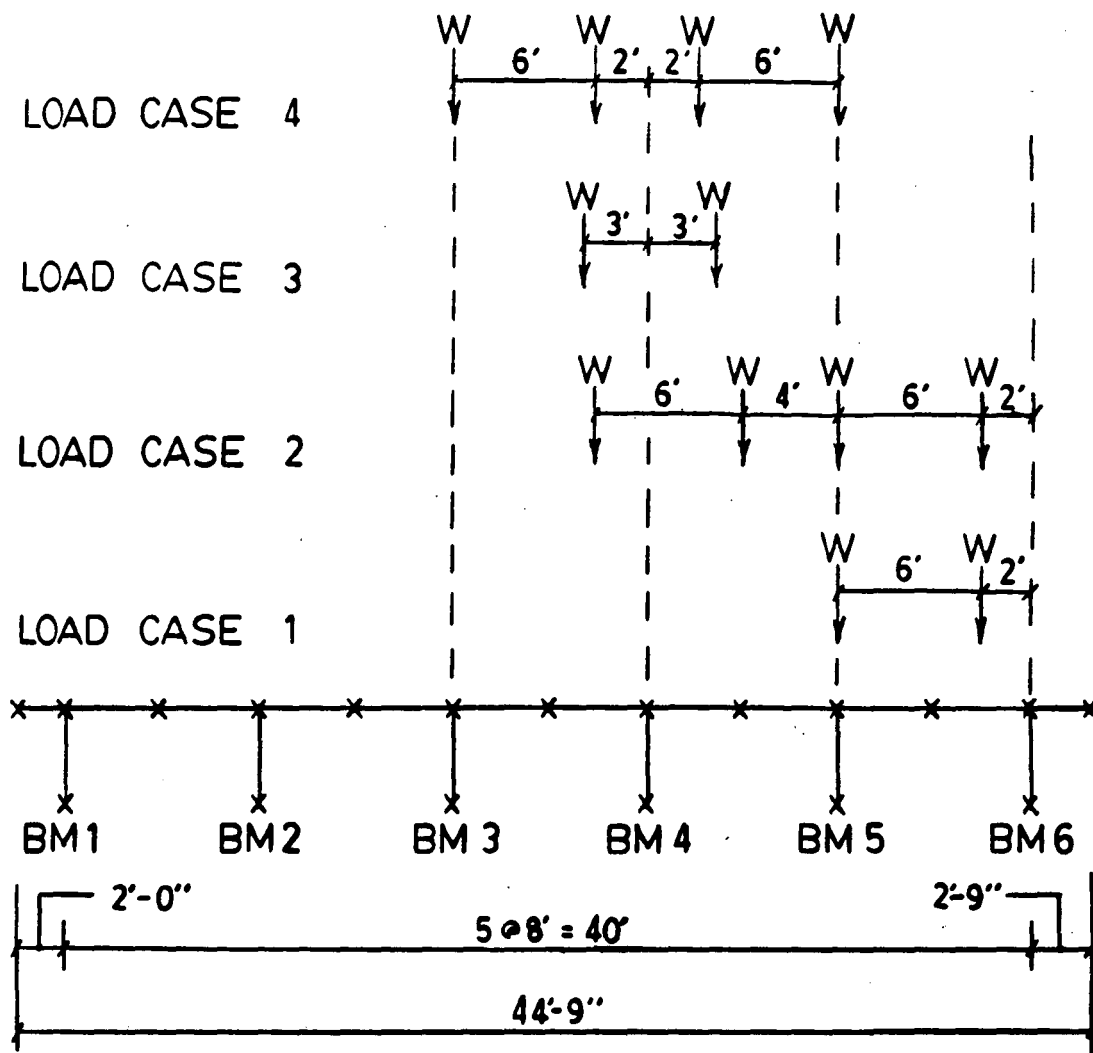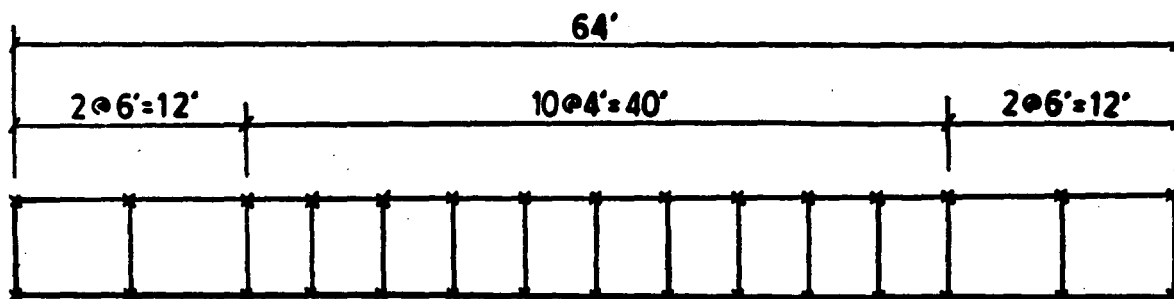
6. Figures

Figure 1: Plan and Section of a Right and a Skew Bridge

LOAD CASE 4

W — 6' — W — 2' — 2' — W — 6' — W

LOAD CASE 3

W — 3' — 3' — W

LOAD CASE 2

W — 6' — W — 4' — W — 6' — W — 2'

LOAD CASE 1

W — 6' — W — 2'

BM 1   BM 2   BM 3   BM 4   BM 5   BM 6

2'-0"   5 @ 8' = 40'   2'-9"

44'-9"

IDEALIZED BRIDGE CROSS SECTION AND LOAD CASES

64'

2@6'=12'   10@4'=40'   2@6'=12'

BRIDGE ELEVATION WITH LONGITUDINAL DIVISIONS

FIGURE 2

MIDSPAN MOMENT DISTRIBUTION
45 SKEW-LOAD CASE #1
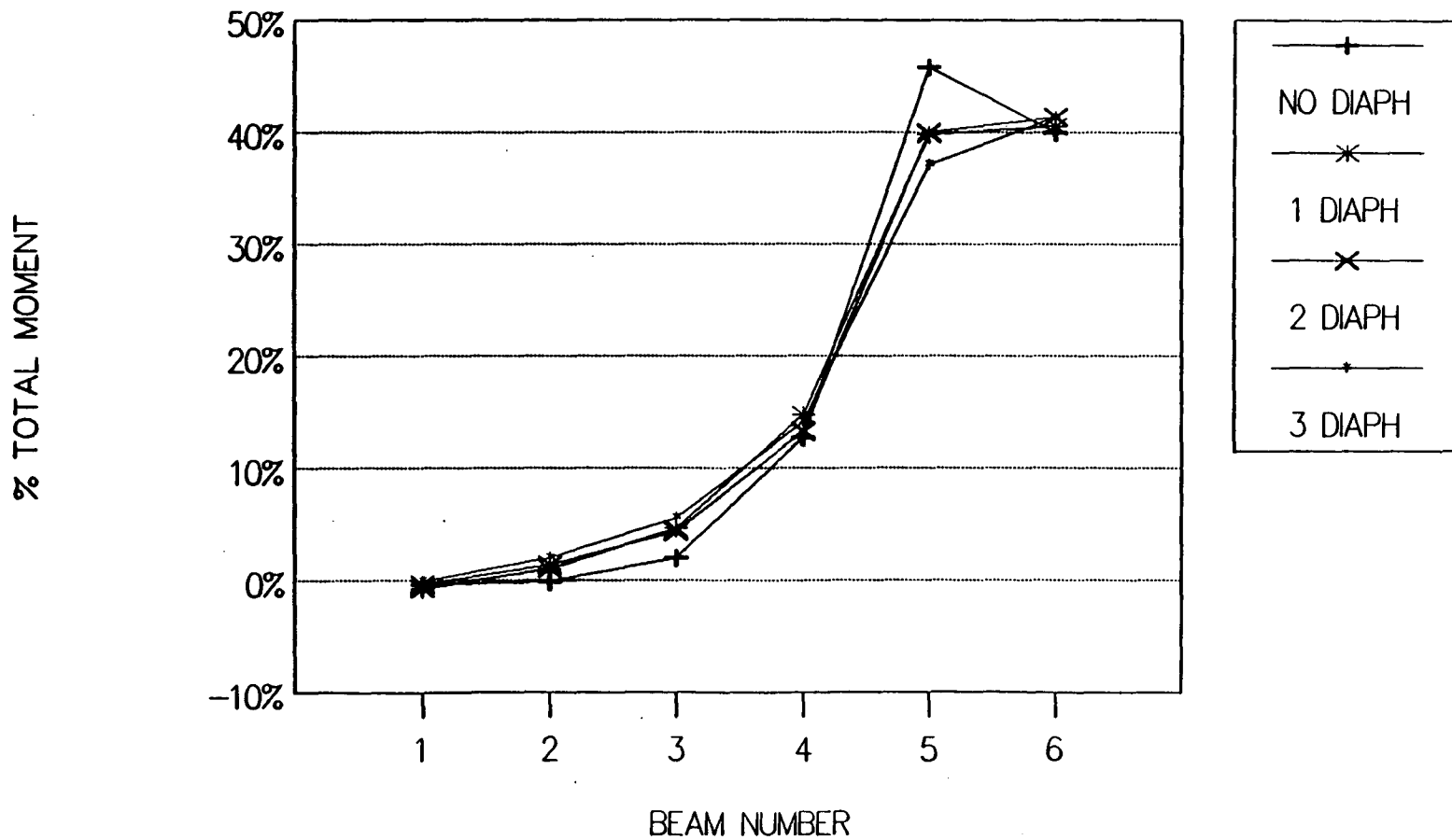
FIGURE 3

MIDSPAN MOMENT DISTRIBUTION
45 SKEW–LOAD CASE #2
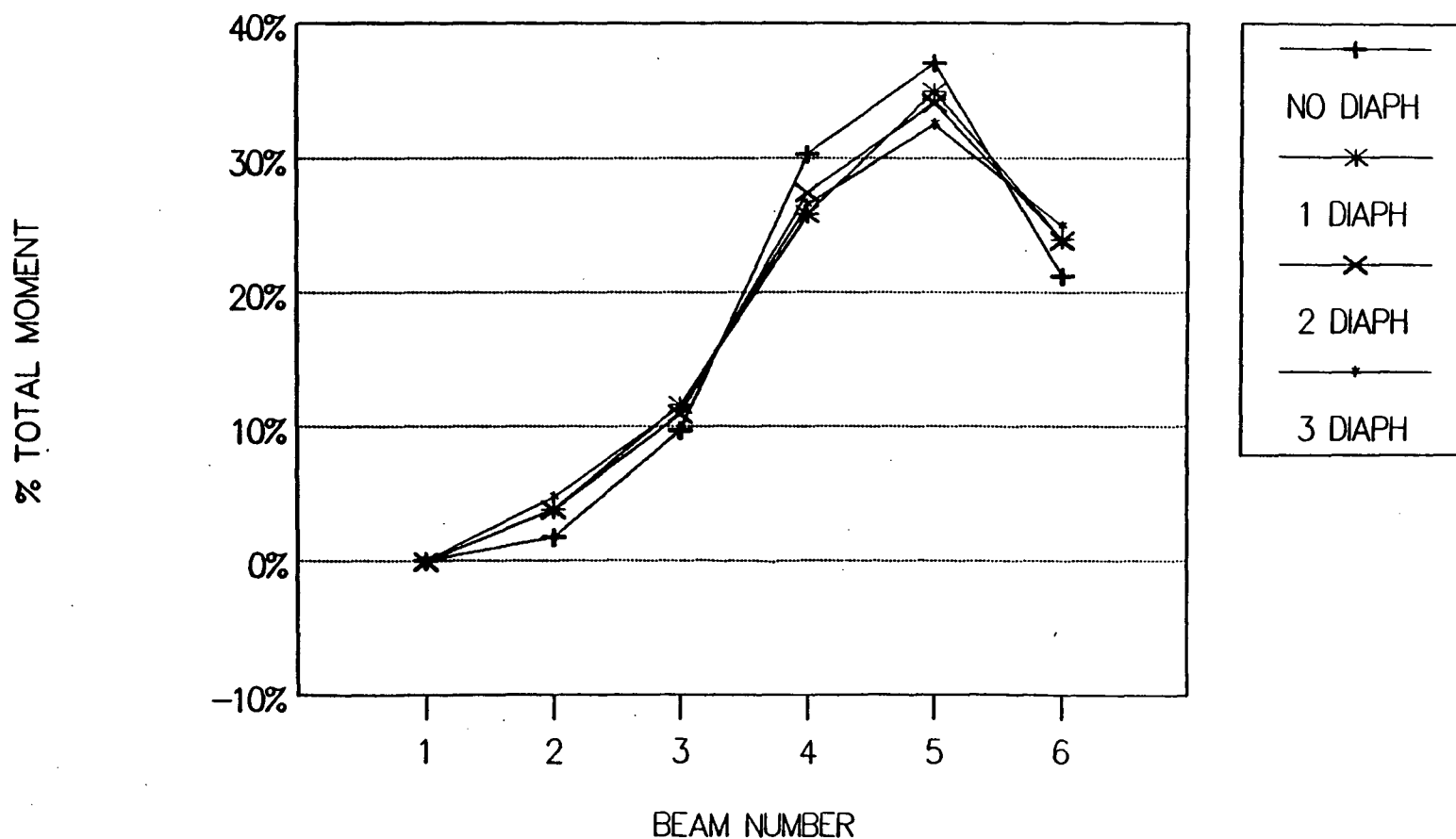
FIGURE. 4

MIDSPAN MOMENT DISTRIBUTION
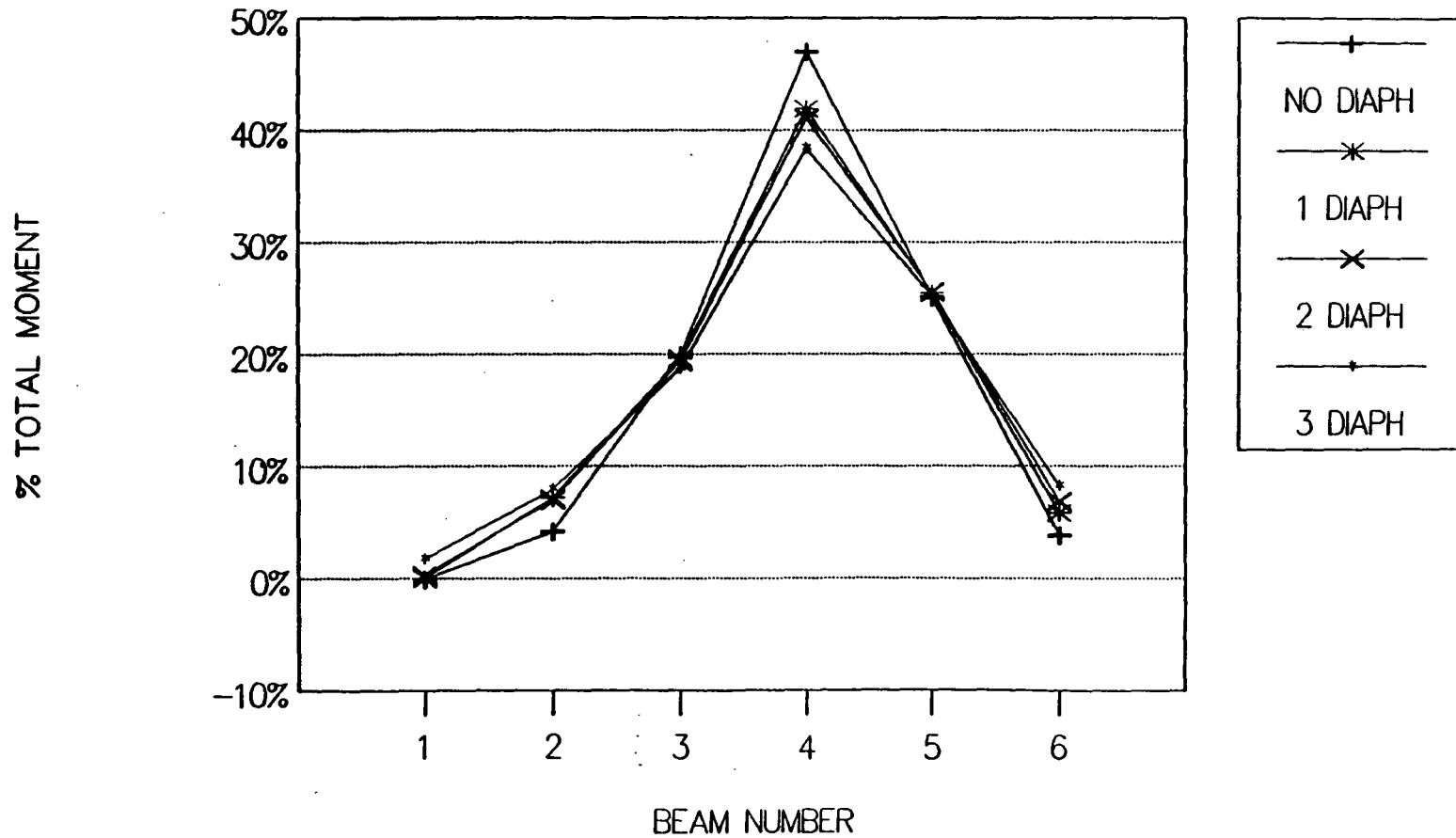# 45 SKEW-LOAD CASE #3

FIGURE 5

MIDSPAN MOMENT DISTRIBUTION
45 SKEW-LOAD CASE #4
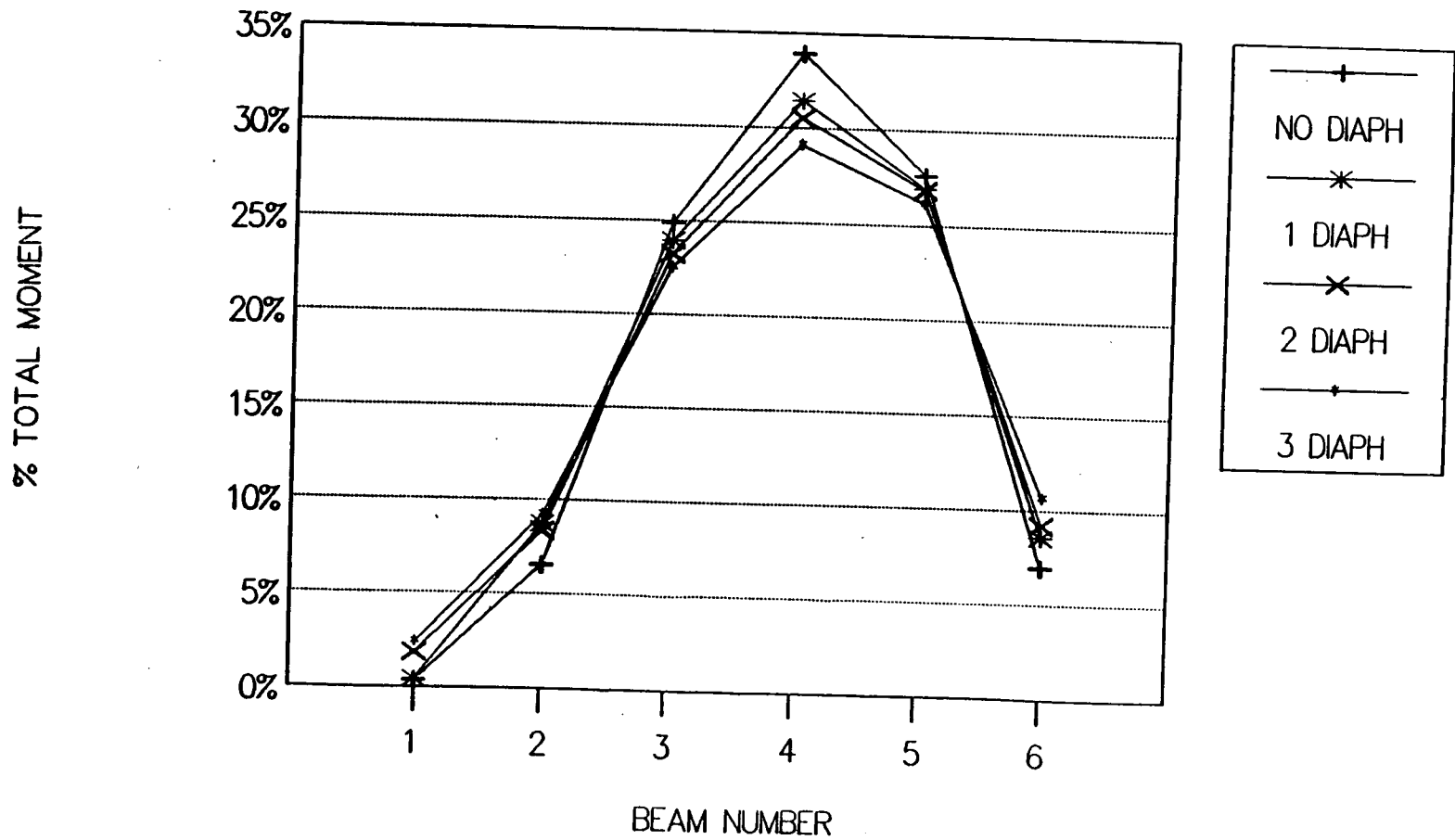

FIGURE 6

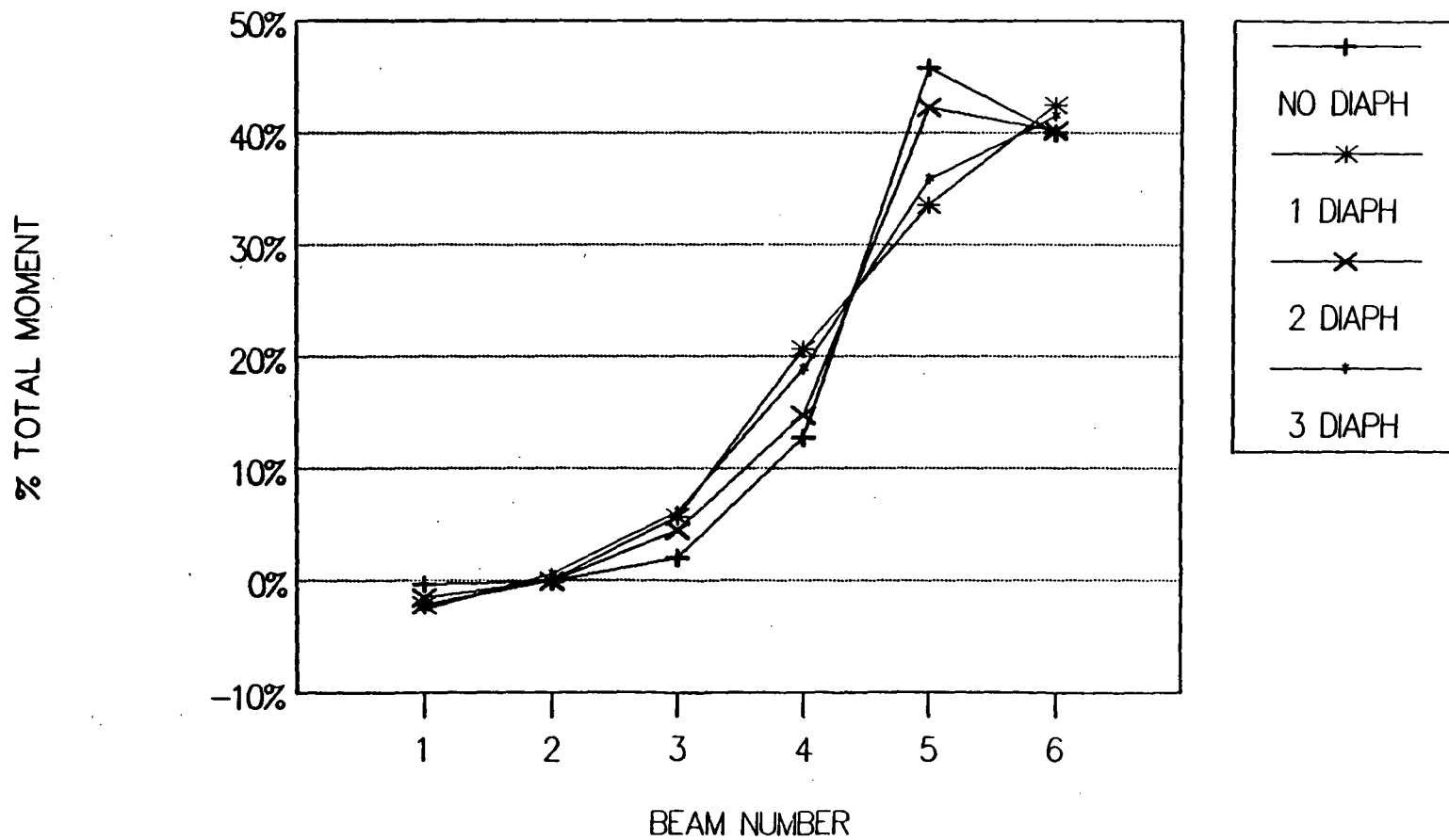# MIDSPAN MOMENT DISTRIBUTION
45 SKEW-LOAD CASE #1 (SKEW DIAPH)

FIGURE 7

MIDSPAN MOMENT DISTRIBUTION
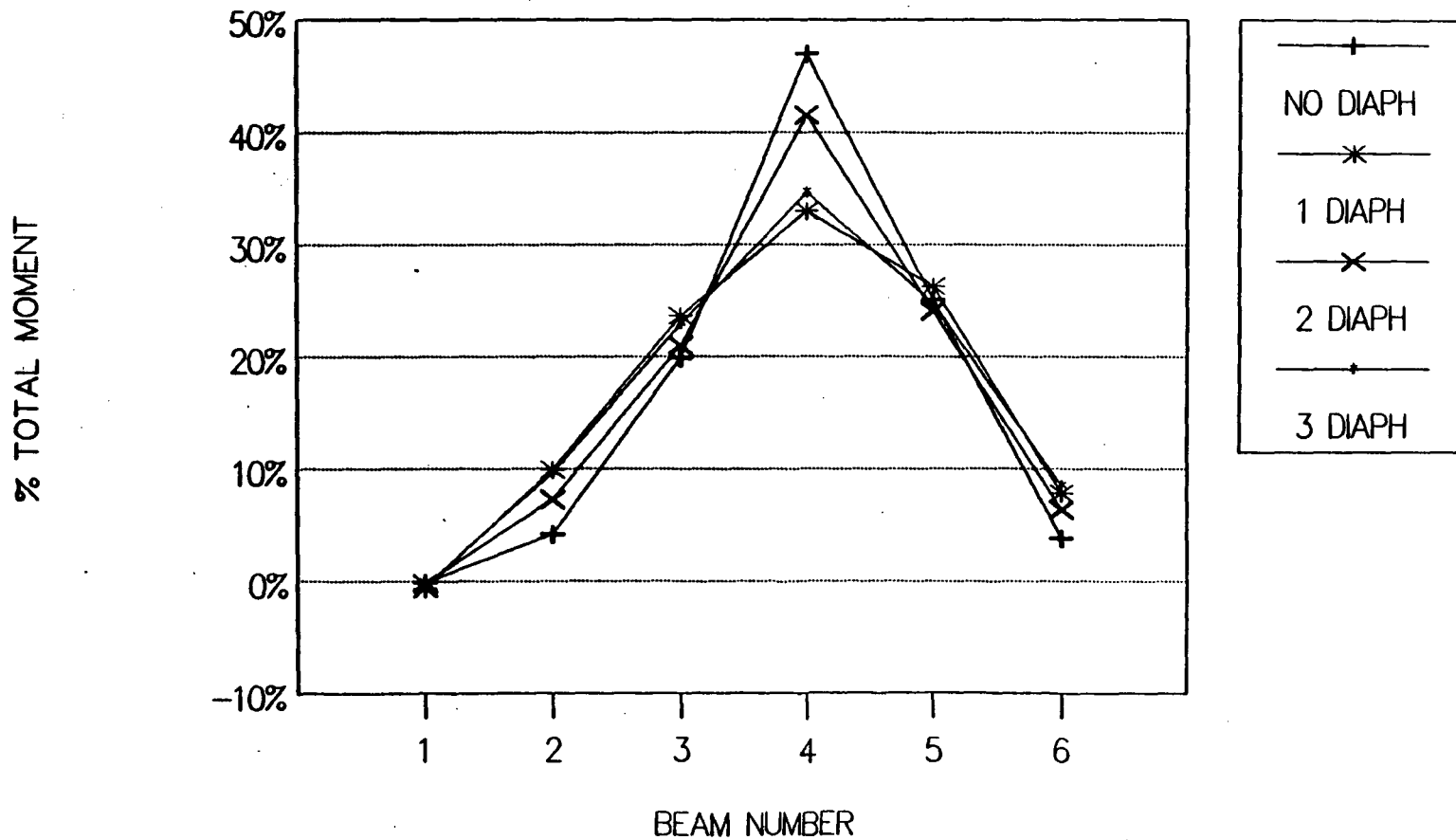# 45 SKEW-LOAD CASE #3 (SKEW DIAPH)

FIGURE 9

MIDSPAN MOMENT DISTRIBUTION

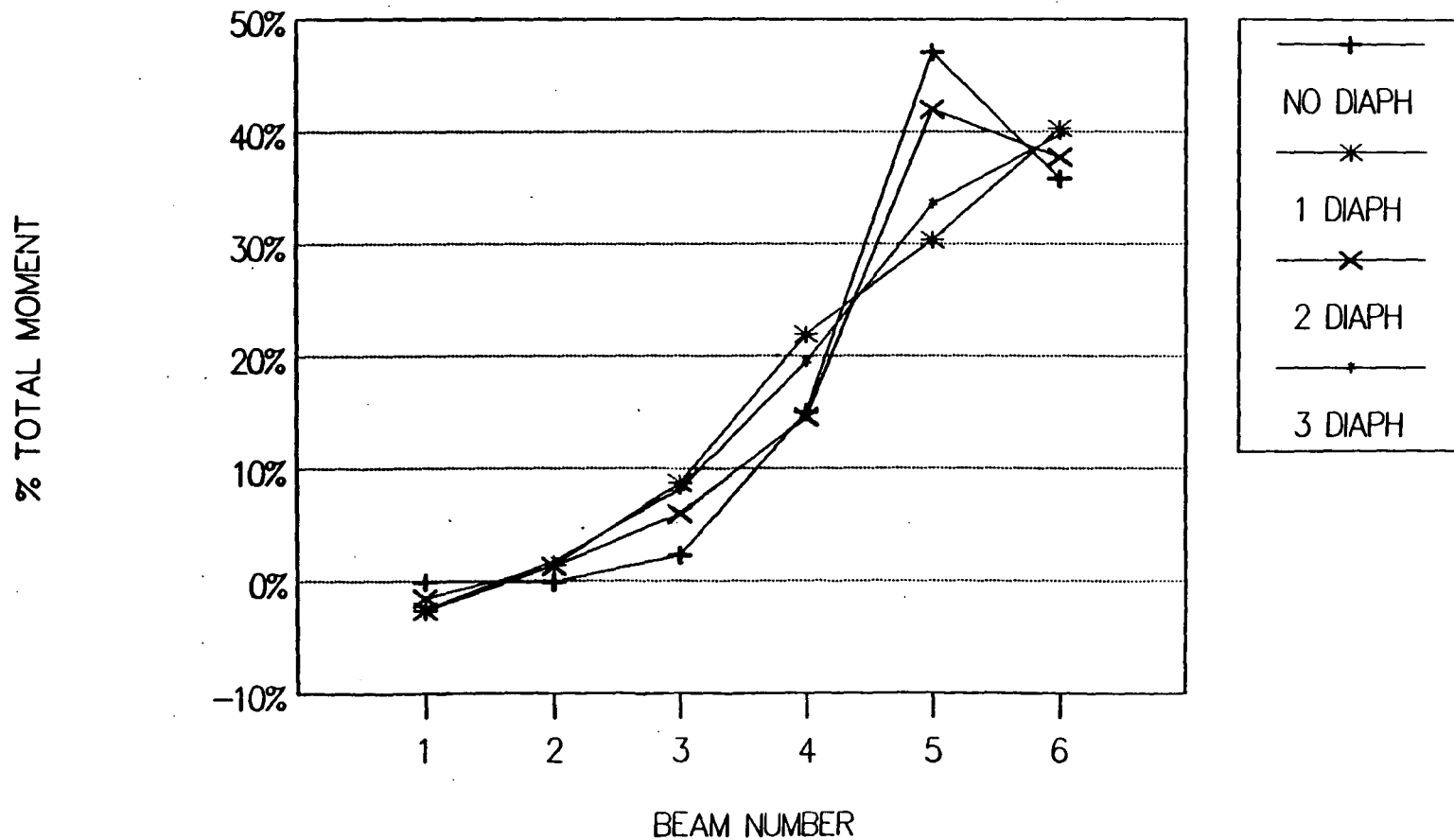# NO SKEW — LOAD CASE #1

FIGURE 9

MIDSPAN MOMENT DISTRIBUTION
NO SKEW – LOAD CASE #3
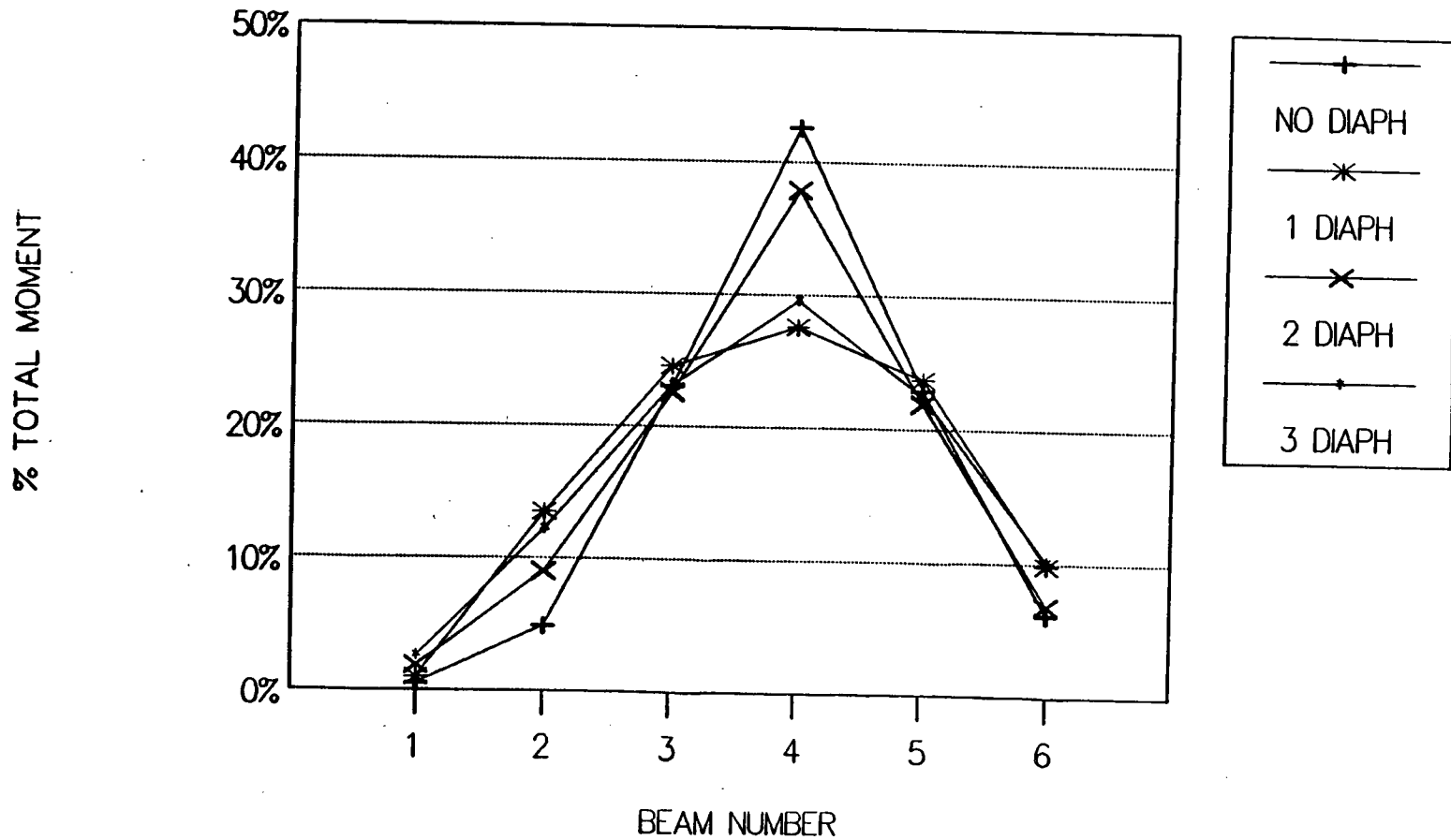

FIGURE 10

MIDSPAN BEAM DEFLECTIONS
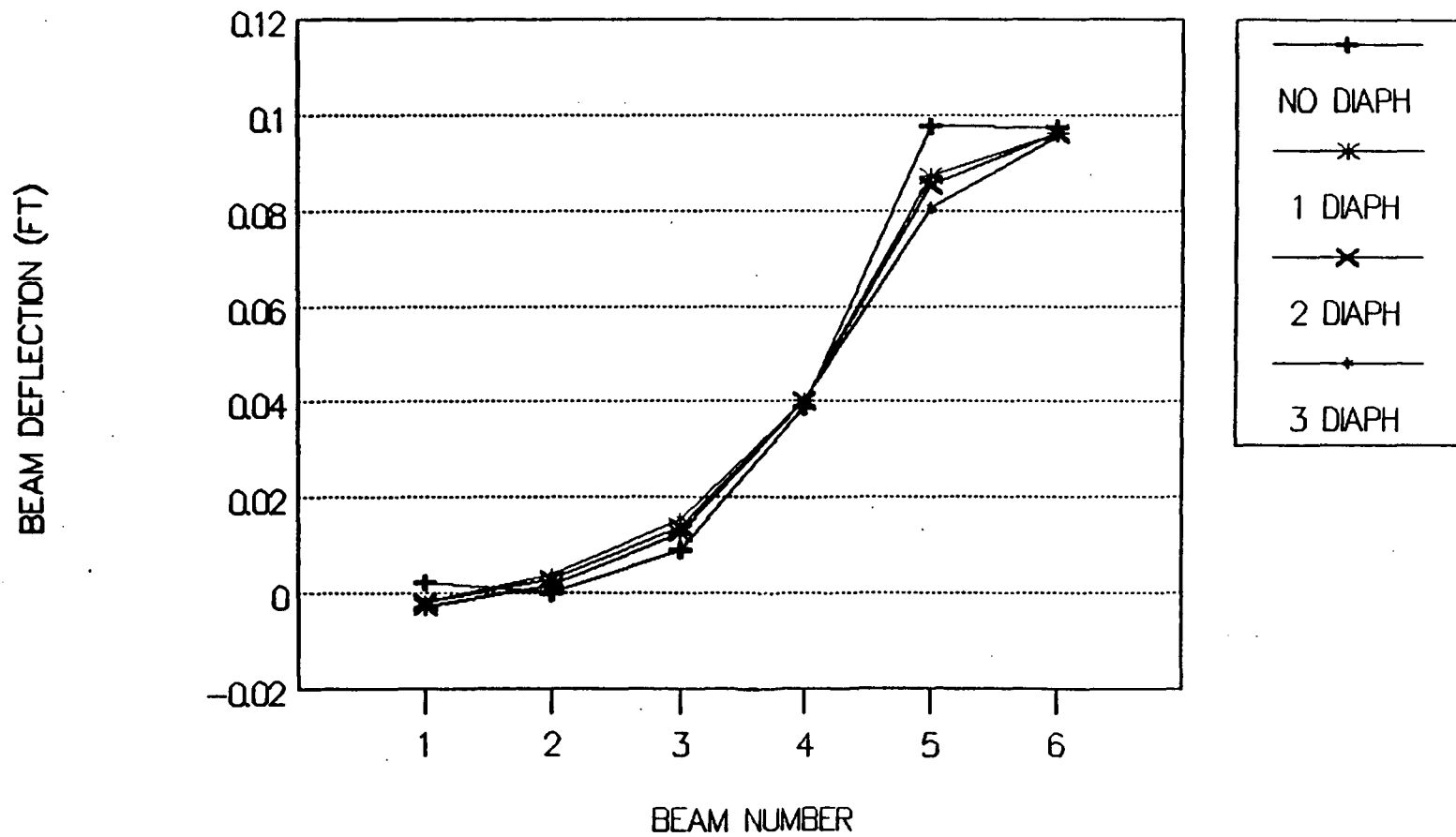# 45 SKEW–LOAD CASE #1

FIGURE 11

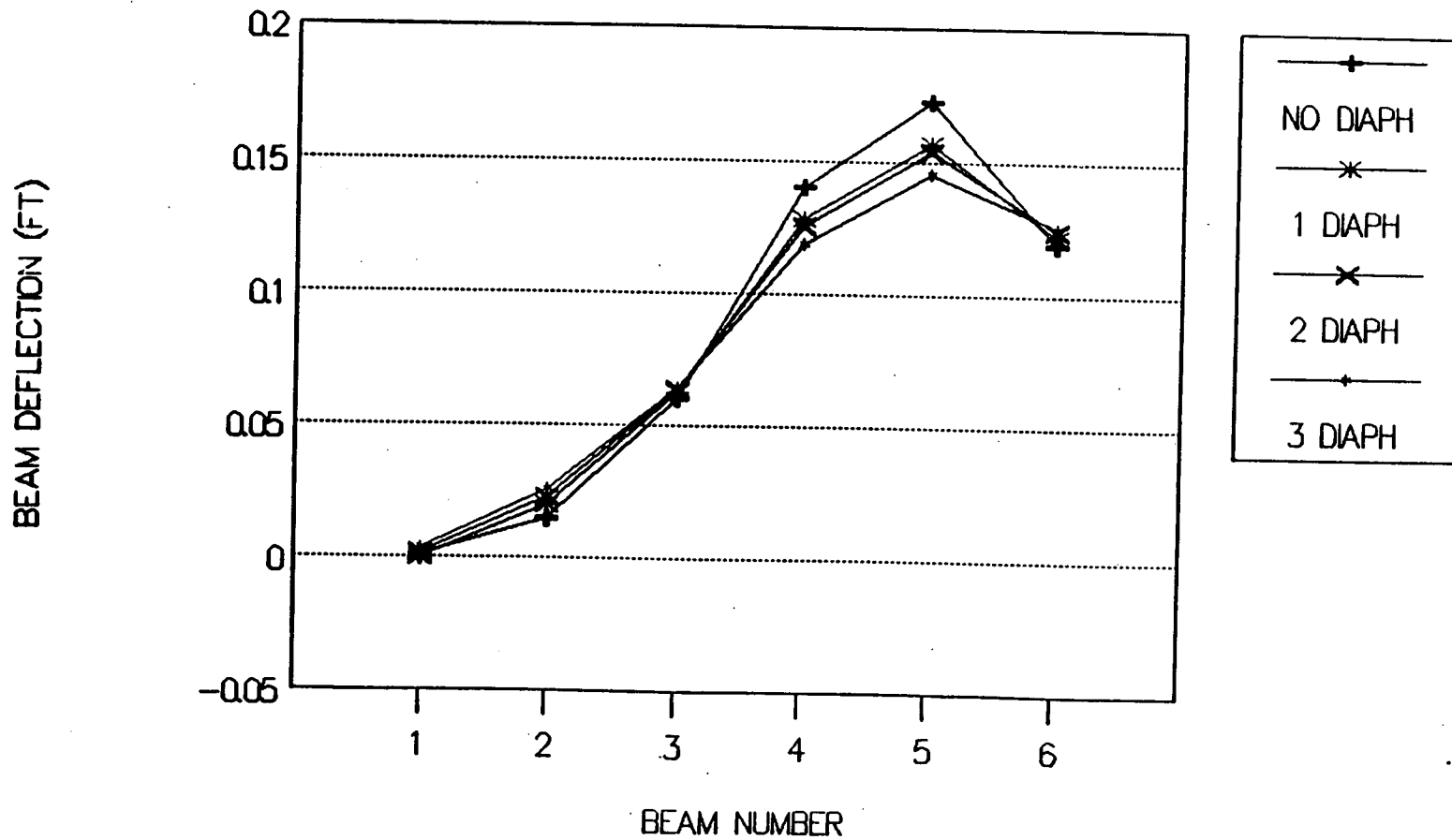# MIDSPAN BEAM DEFLECTIONS
45 SKEW—LOAD CASE #2

FIGURE 12

MIDSPAN BEAM DEFLECTIONS
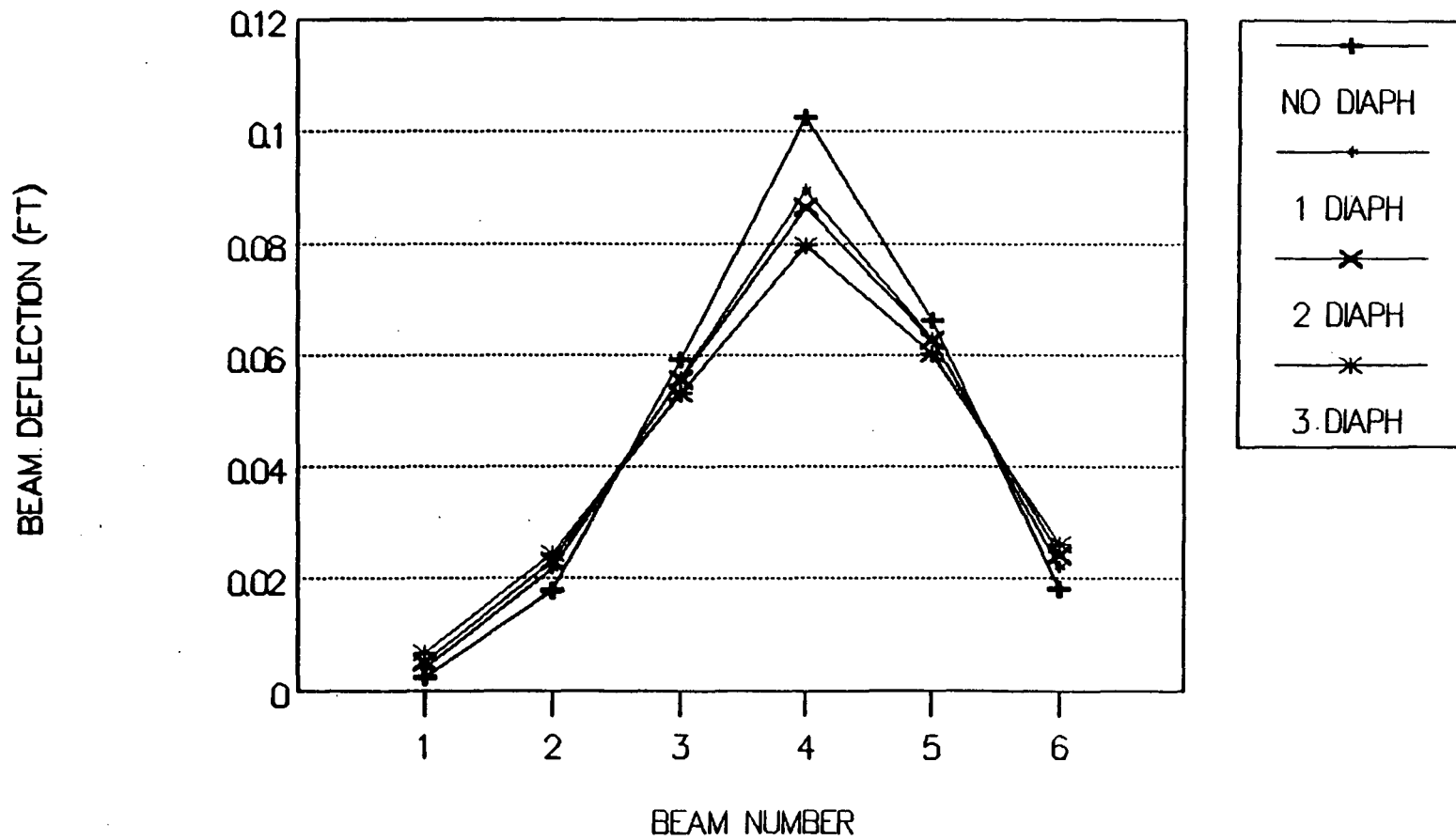# 45 SKEW–LOAD CASE #3

FIGURE 13

MIDSPAN BEAM DEFLECTIONS
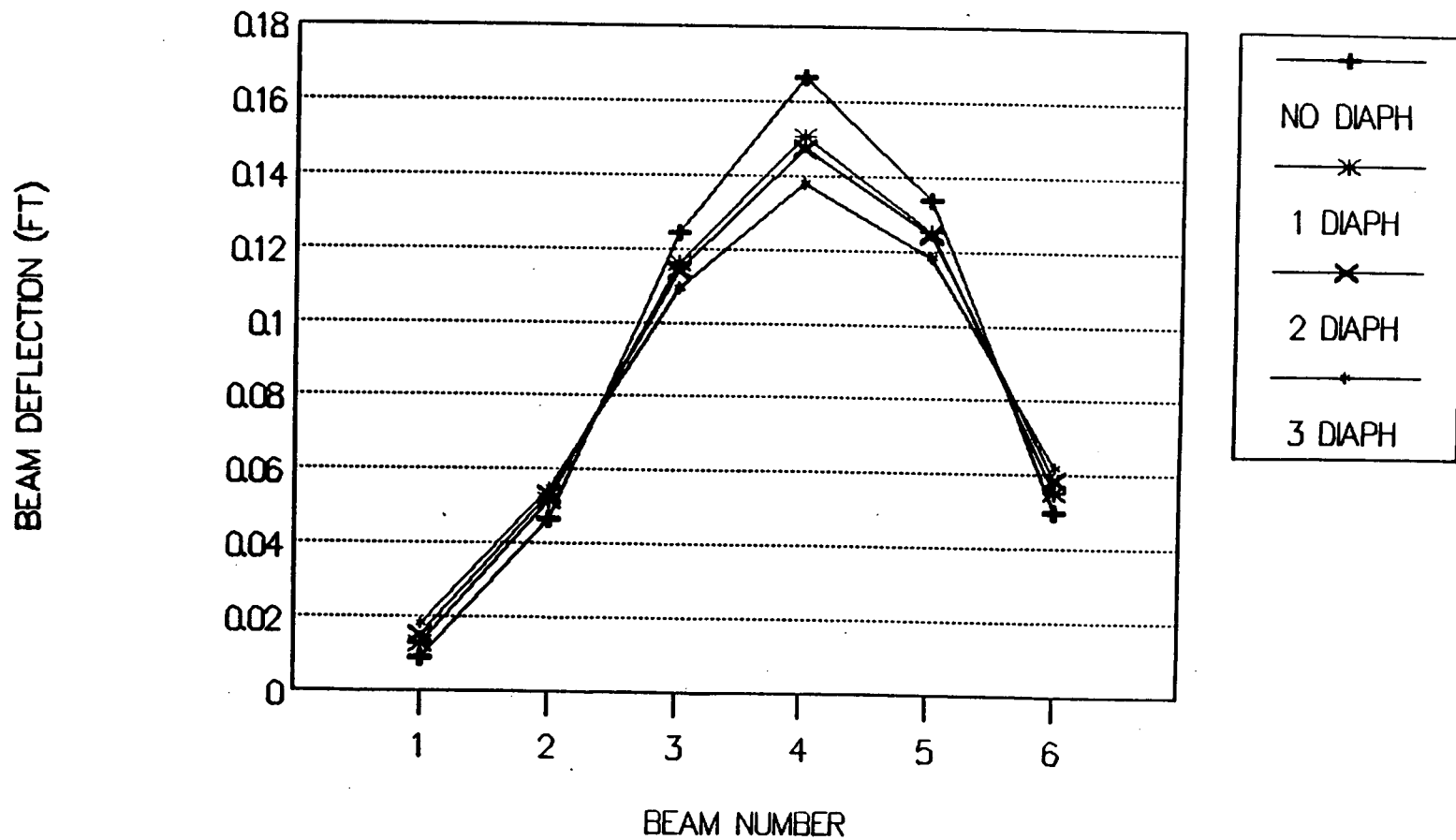## 45 SKEW–LOAD CASE #4

FIGURE 14

MIDSPAN BEAM DEFLECTIONS
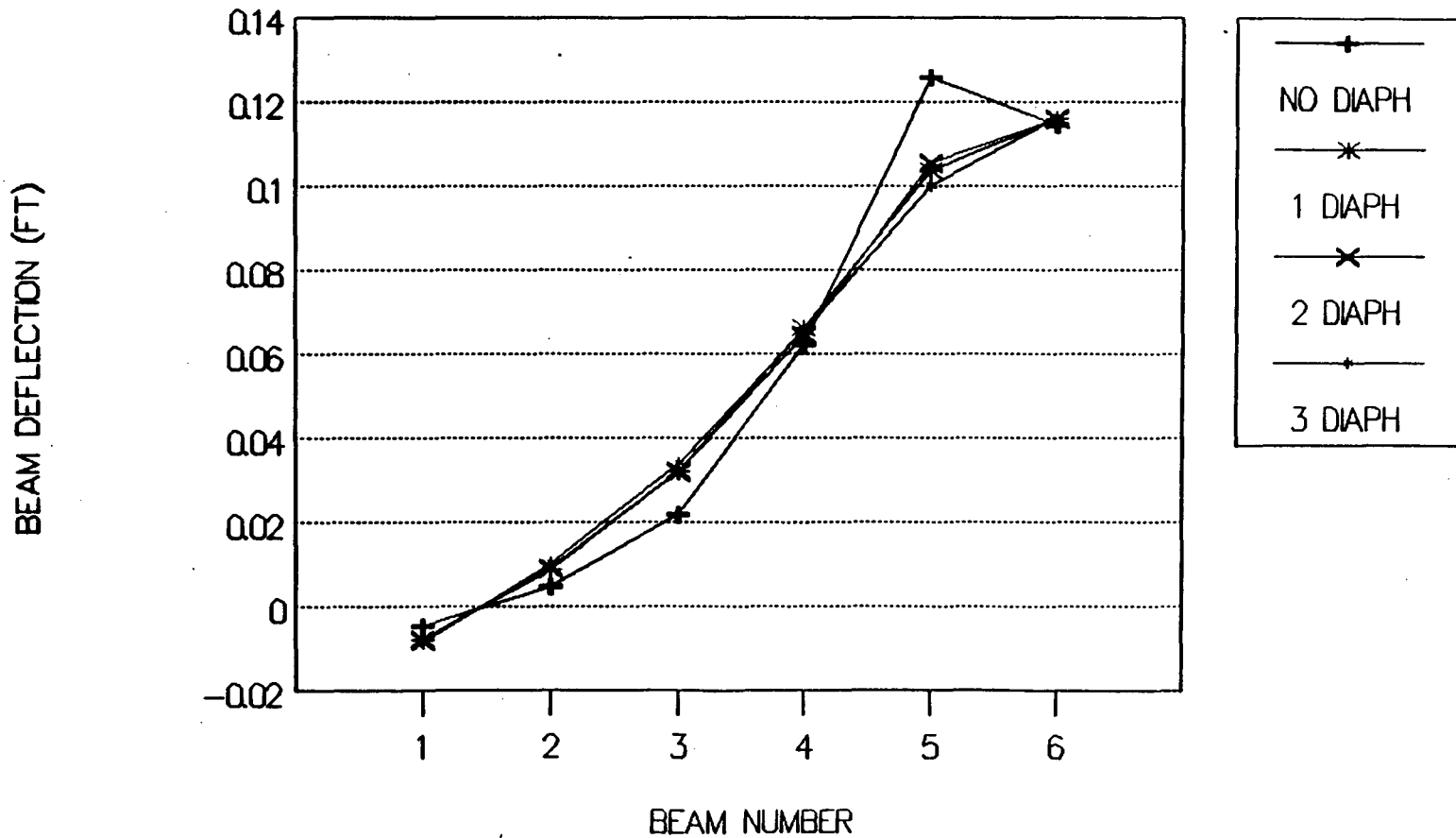# NO SKEW – LOAD CASE #1

FIGURE 15

MIDSPAN BEAM DEFLECTIONS
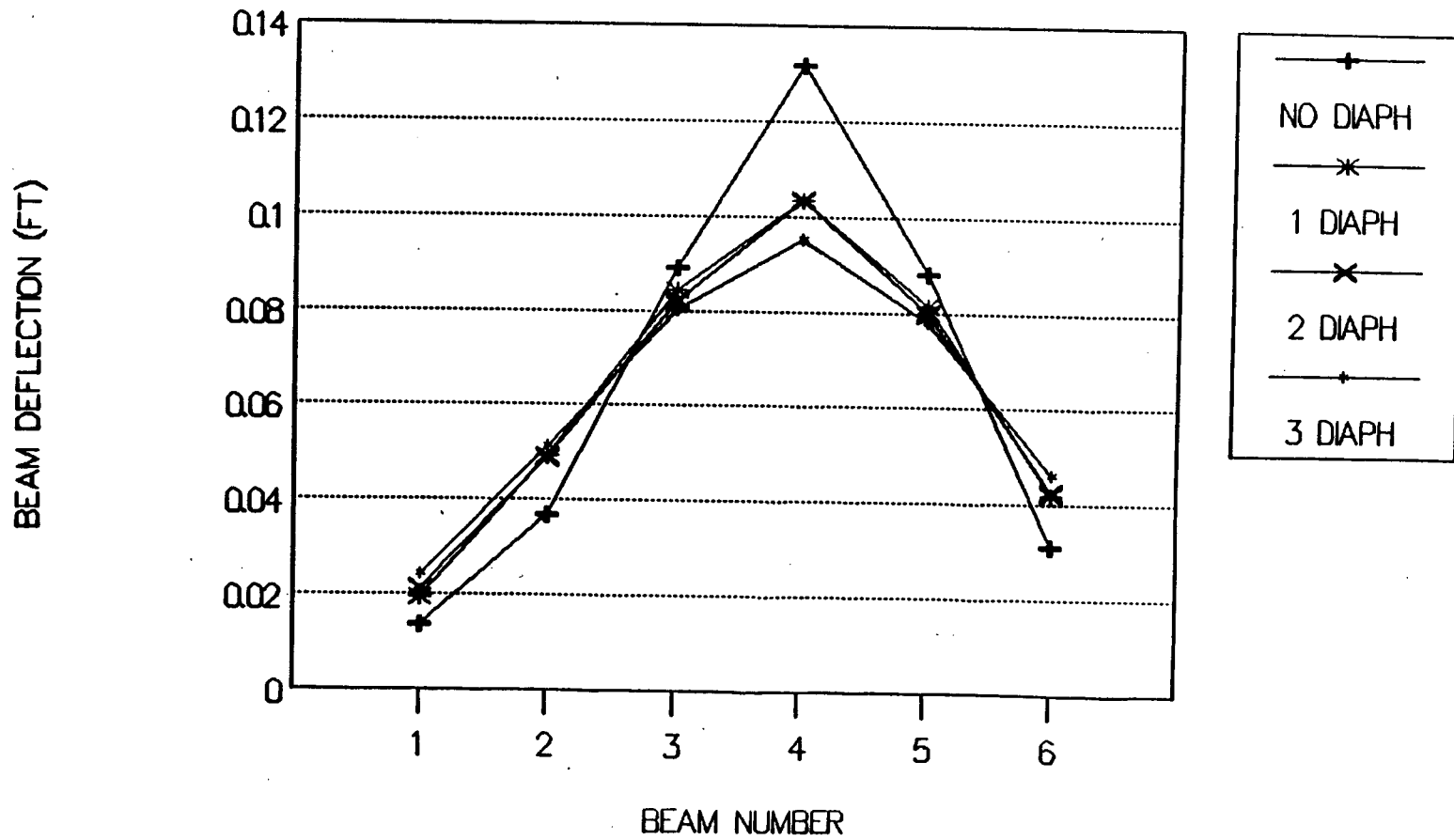# NO SKEW-LOAD CASE #3

FIGURE 16

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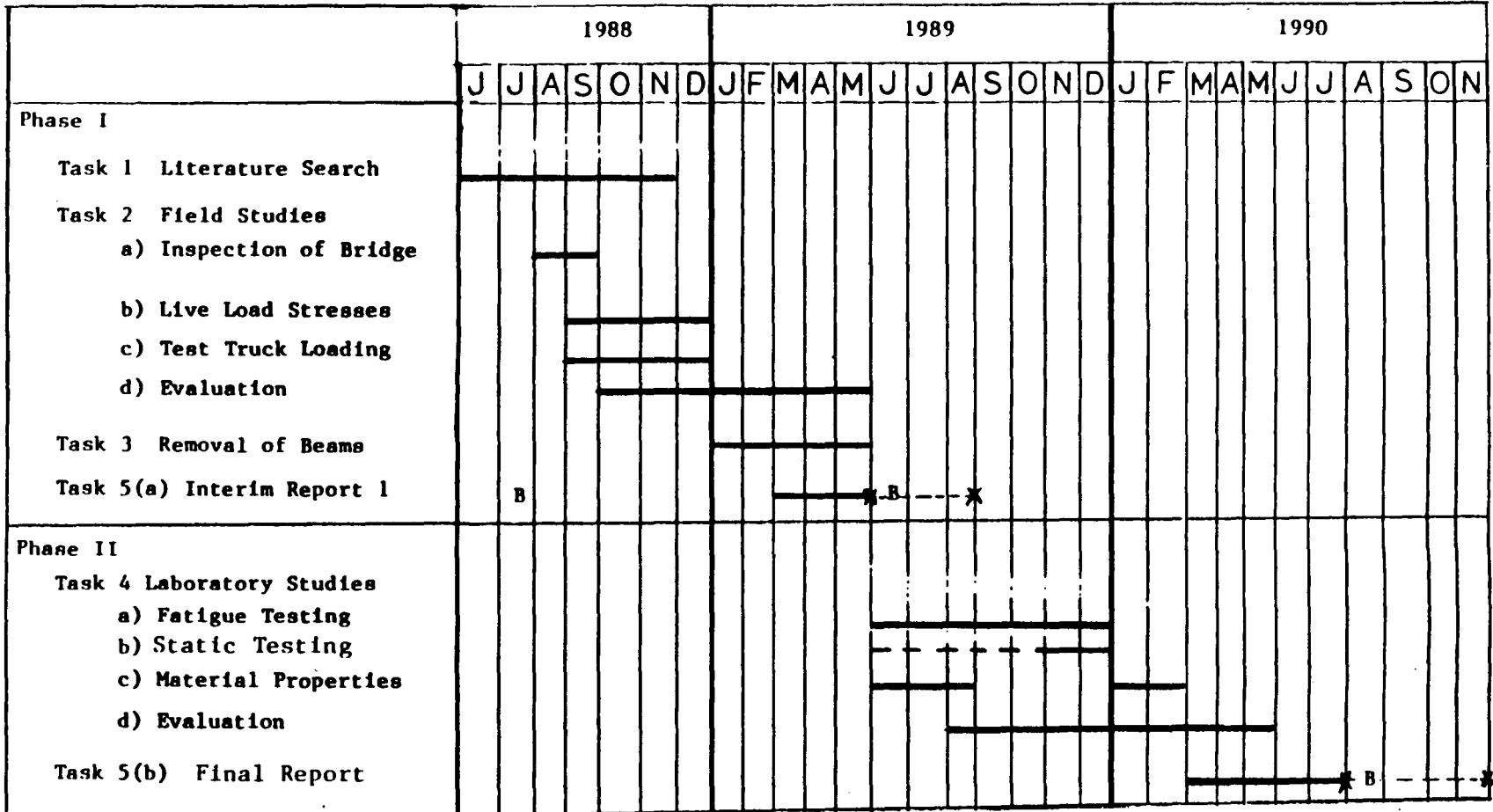   1984

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Appendix A: Project Tasks and Time Schedule

PennDot Research Project 87-04
Fatigue Testing of Prestressed
Beams

Progress Bar Chart

Quarter Ending

	1988	1989	1990
	J J A S O N D	J F M A M J J A S O N D	J F M A M J J A S O N

Phase I

Task 1   Literature Search

Task 2   Field Studies
     a) Inspection of Bridge

     b) Live Load Stresses

     c) Test Truck Loading

     d) Evaluation

Task 3   Removal of Beams

Task 5(a) Interim Report 1

Phase II

Task 4 Laboratory Studies
     a) Fatigue Testing
     b) Static Testing
     c) Material Properties
     d) Evaluation

Task 5(b)   Final Report

B   Briefing Meetings in Harrisburg

Planned ▬▬▬   Actual ─────

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Appendix B: Draft Specification for Field Testing and Cutting and
Removal of Prestressed Concrete Beams

SPECIFICATIONS FOR FIELD TESTING
AND CUTTING AND REMOVAL OF PRESTRESSED CONCRETE BEAMS

Lehigh University, under Pennsylvania Department Of Transportation Research Project No. 87-04, will be conducting load testing of the prestressed concrete beams in the south-bound Ridley Creek Bridge (L.R. 1018, Sta. 505+68.46) during the reconstruction of this bridge. The reconstruction contractor shall cooperate with Lehigh research personnel and perform tasks requested by Lehigh research personnel as described in the following. The cost of such cooperation and tasks shall be included in the reconstruction agreement, payable by PennDOT, and not to be charged to Lehigh University.

For the purpose of contact, the Lehigh research personnel include Dr. Ben T. Yen (Tel. 215-758-3536), Dr. Ti Huang (215-758-3528), Mr. Mark Kaczinski (215-758-3544), or their designated representatives.

TASK NO. 1: FIELD TESTING - TRAFFIC LOAD

1. Near the end of Reconstruction Phase 1, when all three south-bound lanes remain open to normal traffic, Lehigh will conduct a field inspection, install strain gages on prestressed concrete beams and monitor strains under the normal traffic condition.

2. The contractor shall notify Lehigh University personnel at least two months before the estimated completion date of Phase 1.

3. The work by Lehigh research personnel will take approximately eight working days. During this time, complete access to the area beneath the south-bound structure in spans 1 and 2 will be required. The contractor shall cooperate with the Lehigh team and not interfere with its work.

4. The contractor shall provide maintenance and protection of traffic along Sun Drive in accordance with Publication ?. In particular, the contractor shall provide these measures for the Lehigh University instrument van which will be parked on Sun Drive beneath span 1 of the south-bound bridge and adjacent to the pier bent.

5. At the completion of Task 1, all strain gages and connecting wires will remain attached to the beams with suitable protection. The contractor shall exercise caution in subsequent work (through Reconstruction Phase 3) to avoid damage.

TASK NO. 2: FIELD TESTING - TEST TRUCK LOAD

1. At the beginning of Reconstruction Phase 3, after public traffic has been diverted from all three lanes of the south-

bound structure, and before any modification of the south-bound structure, the Lehigh researchers will conduct testing of the structure using special "test trucks". Obtaining and scheduling the "test trucks" will be the responsibility of Lehigh University.

2. The contractor shall notify Lehigh University personnel no later than one week before the installation of maintenance and protection of traffic requirements for Reconstruction Phase 3, and the diversion of south-bound traffic onto the north-bound structure.

3. The work shall take approximately two working days. Throughout this period, the contractor shall ensure that the entire south-bound bridge, as well as approximately 1 mile of roadway on each end of the structure is clear of all construction personnel, materials and equipment.

4. The contractor shall provide maintenance and protection of traffic along Sun Drive in accordance with Publication ?. In particular, the contractor shall provide these measures for the Lehigh University instrument van which will be parked on Sun Drive beneath span 1 of the south-bound bridge and adjacent to the pier bent.

5. The contractor shall not begin the demolition of any part of the south-bound structure until the Lehigh researchers have completed this task.

TASK NO.3: BEAM REMOVAL AND CONCRETE CORES

The contractor shall cut and remove three prestressed concrete beams and obtain 24 concrete core specimens as designated by the Lehigh University researchers after the latter have completed the test truck loading study in Task 2.
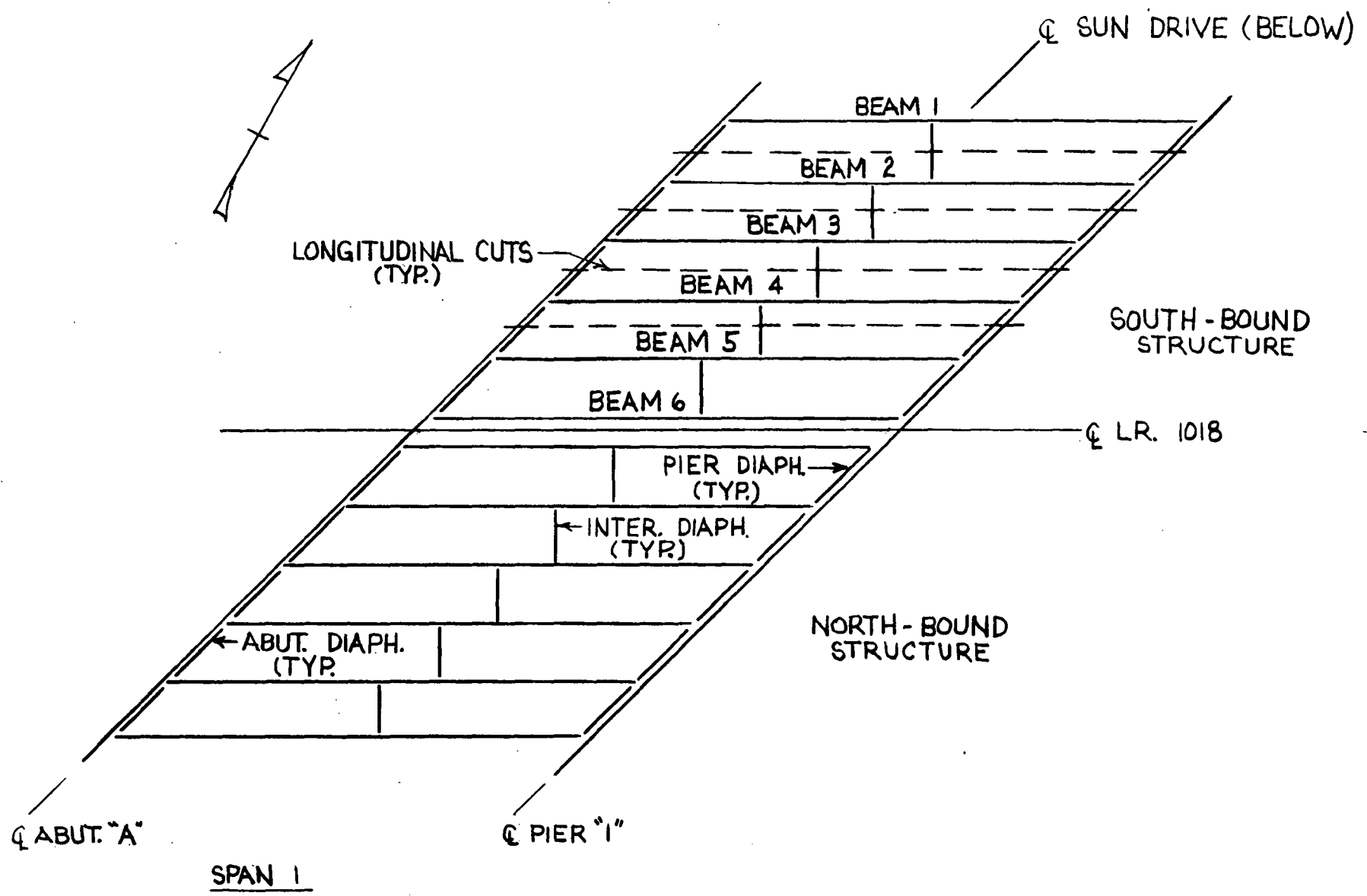
A. Cutting and Removal of Prestressed Concrete Beams

1. Three interior beams (Nos. 2,3 and 4), approximately 64 feet long, in span 1 of the south-bound structure (as shown on the attached sketch) shall be isolated and removed for shipping to Lehigh University for study.

2. The beams shall be isolated by making four longitudinal cuts in the direction of the bridge as shown in the attached plan. The cuts shall be midway between the beams and shall be done in a maner that will result in smooth edge surfaces for the separated T-beams. Sawcutting or an approved equal shall be used. Equipment like a jackhammer shall not be used, as it will damage the concrete and steel adjacent to the cutting.

3. The cutting shall include cuts through both the intemediate and end diaphragms as shown on the attached sketch. The result of cutting shall be three symmetrical T-beams with

approximately 8 foot wide deck slabs and segments of intermediate and end diaphrams. Each of the T-beam specimens will weigh approximately 45 tons.

4. At the conclusion of the testing described in Task 2, a number of strain gages will remain attached to the prestressed concrete beams with appropriate protection applied by Lehigh University personnel. Care shall be exercised in cutting the three designated beams so as not to damage the attached strain gages and connecting wires.

5. While making logitudinal cuts, the contractor shall take caution to laterally brace the isolated beams, in order to prevent tilting. This is particularly important for the fascia beam (No. 1) which is unsymmetrical. (This beam is not needed by Lehigh University for testing. However, its removal from the substructure must be carefully done so as to enable the coring described in Task 3B.)

6. After cutting, the contractor shall lift the three isolated interior T-beams off the substructure and place them on trucks arranged by Lehigh University for transportation. The contractor shall be responsible for the T-beams until they are safely placed on the transporting trucks, after which the responsibility passes to Lehigh University. The contractor shall provide advance notification (at least two weeks) to Lehigh, so that the transportation trucks can be arranged and present at appropriate times.

7. Lifting of the isolated T-beams shall be done by the contractor with great care to avoid damage. Lifting points shall be placed as close as possible to the ends of the beams. Intermediate lifting points shall not be permitted. Lifting shall be done by using cradle-type devices at the bottom of the beams, and tension members passing through small hole drilled through the deck slab. Lifting directly by the deck slab shall not be permitted. Before lifting, adequate bracing of the overhanging deck slab shall be provided at a 8'-0" spacing. (See attached photograph)

B. CONCRETE CORES

1. The contractor shall take twelve 6" diameter cores from the web of the removed fascia beam (No. 1) and twelve 6" diameter cores from the deck slab between beams 5 and 6.

2. The 24 core locations shall be designated by Lehigh University personnel at least one week prior to coring.

3. The coring procedure shall conform to ASTM specification C42-84a.

₵ SUN DRIVE (BELOW)

BEAM 1

BEAM 2

BEAM 3

LONGITUDINAL CUTS
(TYP.)

BEAM 4

BEAM 5

SOUTH-BOUND
STRUCTURE

BEAM 6

₵ L.R. 1018

PIER DIAPH.
(TYP.)

←INTER. DIAPH.
(TYP.)

NORTH-BOUND
STRUCTURE

←ABUT. DIAPH.
(TYP.

₵ ABUT. "A"

₵ PIER "1"

SPAN 1

PARTIAL FRAMING PLAN - SPAN 1
N.T.S.