AN INVESTIGATION OF SHEAR STRENGTH OF PRESTRESSED CONCRETE AASHTO TYPE II GIRDERS

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ABSTRACT

An extensive load test program was conducted to investigate the shear strength of prestressed concrete girders. A total of 33 full-size AASHTO type II prestressed girders were load-tested at the Florida Department of Transportation's Structures Research Center in Tallahassee. The precast girders were fabricated and transported by DURASTRESS of Leesburg, Florida as part of the contribution of the prestressed concrete industry.

The main parameters in the investigation were the percentage of shielded strands, the size of prestressing strands, and the web shear reinforcement. The main objectives were to study transfer and development lengths of prestressing strands and the shear strength of the girders.

This report concentrates mainly on the shear strengths of the specimens. The test shear strengths are compared with predictions based on the AASHTO 1989 Standard Specifications for Highway Bridges, the 1990 and 1991 AASHTO Interim Specifications. Also, test results were compared to the recently proposed AASHTO Code revisions for shear design, which are based on the Modified Compression Field Theory (MCFT).

The results indicate that the current AASHTO code provisions predict shear strengths better than the new proposed revision (i.e., MCFT). The investigation shows that although a computer program based on the MCFT provided a good approximation of shear strengths, the crude application of the MCFT, using tables as suggested in the proposed code, leads to inaccuracies in computing shear strength. The principal conclusion of the research is that the proposed code provisions for shear are not justified since the current AASHTO code consistently provided the best approximation of the measured shear strength.

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contribution of the Prestressed Concrete Industry of Florida to this reciated. These contributions include the manufacture and transportation tructures Research Center by DURA-STRESS Inc. The extent of their rch project is a model for other industries.

Mr. G. Kent Fuller, Vice President of DURA-STRESS Inc. through out eciated.

ressed to the entire staff of the Structures Research Center, Florida tation, and particularly to Dr. M. Issa, Ms. Sandi Curlee, Mr. Adnan ton, Mr. Frank Cobb, Mr. Randy Bradley, Mr. Alex Fishburn, Mr. Ron in and to many other employees who are no longer with the Structures their help this project would not have been possible.

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ouragement of Mr. Tony Garcia, Mr. Jerry Potter, Mr. Bill Deyo and ly appreciated.

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NOTATIONS

A_c = area of concrete member, sq.in.

A_p = area of a single prestressing strand, sq. in.

A_{ps} = area of prestressing steel, sq. in.

A_{px} = total area of longitudinal prestressing tendons, sq. in.

A_{sx} = total area of non-prestressed longitudinal reinforcement, sq. in.

A_s = area of non-prestressed tension reinforcement, sq. in.

A_s^* = area of prestressing steel, sq. in.

A_v = area of shear reinforcement within a distance s, sq. in.

a = depth of equivalent rectangular stress block, in.

a = shear span, distance between concentrated load and centerline of support, in.

b = effective slab width, in.

b_w = web width or diameter of a circular section, in.

b_v = width of cross-section at contact surface being investigated for horizontal shear

c_x = maximum vertical distance from any location in the concrete to longitudinal reinforcement, in.

c_v = maximum horizontal distance from any location in the concrete to transverse (shear) reinforcement, in.

d = effective flexural depth, being the distance from extreme compression fiber to the centroid of tensile force, in.

d_{bx} = diameter of longitudinal reinforcement, in.

d_{bv} = diameter of transverse (shear) reinforcement, in.

d_v = jd, effective shear depth, which can be taken as the distance measured perpendicular to the neutral axis, between the resultants of the tensile and compressive forces due to flexure, but need not be less than the greater of 0.9d or 0.72h, in.

E_c = modulus of elasticity of concrete, psi

E_p = modulus of elasticity of prestressing tendons, psi

E_s = modulus of elasticity of reinforcing bars, psi

f_1 = diagonal tensile stress, psi.

f_2 = diagonal compressive stress in the concrete, psi.

f'_c = specified compressive strength of concrete at 28 days, unless another age is specified, psi

f'_{ci} = compressive strength of concrete at the time of initial prestress

f_p = stress in prestressing steel, psi

f_{ps} = average stress in prestressing steel at nominal resistance of member, psi

f'_s = ultimate strength of prestressing steel, psi.

f_{se} = effective stress in the prestressing steel after losses, psi

f^*_{su} = average stress in prestressing steel at ultimate load, psi.

f_v = stress in stirrups, psi.

f_x = stress in longitudinal bars, psi.

f_y = specified minimum yield strength of reinforcing bars, psi

h = overall thickness or depth of a member, in.

I = moment of inertia about the centroid of the cross section, in^4.

k_1 = coefficient, equal to 0.4 for deformed bars or 0.8 for plain or bonded strands

L = span length, ft.

L_d = distance from end of girder to point of application of load, in.

M_u = factored moment at a section, in. lbs.

N = north end

N_c = factored tensile force applied at top of a bracket or corbel, acting simultaneously with V. to be taken as positive for tension, lbs.

N_u = factored axial load normal to the cross section, occurring simultaneously with V_u; includes effects of tension due to creep and shrinkage, lbs.

P_j = jacking force of prestressing strands, lbs.

S = south end; or average stringer spacing in feet

s = spacing of shear spacing of reinforcing bars, in.

s_{mx} = crack spacing indicative of the crack control characteristics of longitudinal reinforcement, in.

s_{mv} = crack spacing indicative of the crack control characteristics of transverse reinforcement, in.

$s_{m\theta}$ = spacing of inclined cracks, in.

s_x = spacing of longitudinal steel, in.

s_v = spacing of transverse steel, in.

v = shear stress, psi

V = shear force at a section, lbs. or kips

V_c = nominal shear resistance provided by concrete, lbs.

V_{ci} = nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment

V_{cw} = nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in the web

V_D = shear force due to dead loads

V_{L+I} = shear force due to live load and impact

V_n = nominal shear strength of section, lbs.

V_p = component in the direction of the applied shear of the effective prestressing force, positive if resisting the applied shear, lbs.

V_s = factored shear resistance provided by shear reinforcement, lbs.

V_u = factored shear force at section, lbs.

α = angle between inclined shear reinforcement and the longitudinal axis of member, degrees

β = a coefficient used to compute the shear resisted by tensile stresses in the concrete

β_1 = factor for concrete strength; taken as 0.85 for concrete strengths, f'c, up to and including 4,000 psi. For strengths above 4,000 psi, the factor is reduced continuously at a rate of 0.05 for each 1,000 psi but shall not be less than 0.65.

ϕ = strength reduction factor; or resistance factor

θ = inclination of the diagonal compressive stresses, degrees

ρ^* = A_s^*/bd, ratio of prestressing steel

ρ_v = $A_v/b_w s$, shear steel ratio

ρ_v = $(A_{sx} + A_{px})/A_c$, longitudinal steel ratio

ϵ_x = longitudinal strain at mid-depth of member when the section is subjected to M_u, V_u, and N_u, positive when tensile

γ^* = factor for the type of prestressing steel; equals 0.28 for low-relaxation steel

CHAPTER 1.

INTRODUCTION AND BACKGROUND

The current popularity of prestressed concrete in bridge construction is likely to increase in the future. In the design of such girders it is usual to aim for maximum eccentricity of the prestress force where the bending moment is largest. In a simply supported beam, this point is located at or near midspan, and the required eccentricity at the supports is considerably less. It has been customary to drape or harp the tendons in order to avoid the occurrence of unduly high tensile stresses near supports. More recently, the debonding of strands near supports has been used with the same result. The use of such "shielded" or "blanketed" strands allows the use of straight strands throughout the beam, and has been found to be convenient, economical and safe. Therefore, the prestressing industry has widely adopted the use of shielding, over draping or harping of strands.

The adequacy of the current provisions in the ACI and AASHTO codes with respect to transfer and development lengths as well as the shear strength has been the source of much controversy. An attempt to introduce a new AASHTO Code is underway. A comprehensive series of studies is now in progress in North America in order to provide answers to the questions raised.

The Florida Department of Transportation (FDOT) recognizes its responsibility to the public to design and construct efficient, economical, and durable structures. Prestressed concrete girders are used frequently in Florida's bridges. Therefore, ensuring that these bridge components are efficiently designed is of great importance to the Department. During the past three years FDOT's Structures Research Laboratory has been involved in the load testing of a large number of full-scale AASHTO Type II prestressed concrete girders.

The main objectives of this study were to determine experimentally the actual values of transfer and development lengths of prestressing strands, effect of strand shielding (debonding) on development length, shear and fatigue behavior, and the shear strength as it compares to existing and proposed code provisions.

This shear capacity study is particularly significant in light of proposed changes to the AASHTO code for the design of members subject to shear and torsion. This report presents the results of the results of the shear capacity investigation.

Thirty three (33) full-scale AASHTO type prestressed concrete girders, designed in accordance with the AASHTO 1991 Interim Specifications were fabricated and tested. The girders were divided into three groups according to the size of strands used (1/2", 1/2" special and 0.6" diameter strands). All girders were designed for approximately the same nominal flexural moment.

In this report the test results are presented and compared with predictions based on the 1989 AASHTO Standard Specifications for design of highway Bridges[1] ,the 1990[2] and 1991[3] Interim Specifications of that code, and the proposed revisions[4] of the code based on the Modified Compression Field Theory (MCFT)[5].

The MCFT was applied using the procedures described in Reference 4, which provides tables for determining the quantities, R and 0 in the MCFT equations. The term R is a factor used in computing the shear resistance due to the tensile stresses in the concrete, and \ 0 is the angle of inclination of the diagonal compressive stresses. Before using the tables, it is necessary to calculate the longitudinal strain, ε_x, using a prescribed equation (Eqn. 5.8.3.4.2-2 of Ref. 4). Alternatively, a value of $\varepsilon_x = 0.003$ may be assumed for prestressed members. Both approaches were used in the comparison with test results. In addition, shear strength values were calculated using a computer program "RESPONSE" as presented and documented by Collins and Mitchell (1991) (Ref. 5). This program is based on the MCFT.

The required shear capacities were computed for AASHTO type If girders of a hypothetical bridge spanning 40 ft. with girders spaced at 10 ft on center. A 42 in. wide by 8 in. thick composite concrete slab which was cast on the top of each girder prior to testing. In calculating the loads carried by an interior beam, an overall roadway width of 43 ft was assumed, which is the width of a typical two lane interstate ramp or a divided highway. The bridge girders were designed to carry the composite dead load of the barriers and future surfacing, an 8 in. concrete deck and the required live load (HS-20 truck). In accordance with the Florida Department of Transportation's (FDOT) Structures Design Guidelines[6], the live load was increased by five percent (5%) to account for intermediate diaphragms. The required shear capacity was determined from the shear loads computed using the FDOT's Prestressed Concrete Girder Design Program[7] (PCGDP).

In comparing the various predicted shear capacities with the test results, plots are used to show the following:

1) Required capacity, Vu/0.85, due to the loading;
2) Shear capacity provided based on the current AASHTO Code [1,2,3] (Vn CURRENT AASHTO);
3) Shear capacity provided based on shear provisions of the proposed AASHTO Code[4]:
   a) Using calculated values of longitudinal strain, ε_x, (V_n PROPOSED AASHTO);
   b) Assuming longitudinal strain, $\varepsilon_x = 0.003$, (V_n AASHTO ($\varepsilon_x = 0.003$);
4) Shear capacities as computed by computer program "RESPONSE", (V_n RESPONSE); and
5) The maximum shear load measured from the girder tests, (TEST NORTH and TEST SOUTH).

CHAPTER 2

DESCRIPTION OF SPECIMENS AND INSTRUMENTATION

2.1 DETAILS OF TEST PROGRAM

The test program consisted of thirty three (33) AASHTO Type H prestressed concrete girders, 41 feet long. All beams were designed for approximately the same ultimate flexural strength (2100 ft-K). Three different size 270 ksi, Low Lax prestressing strands were used in the investigation; namely, 1/2", 1/2" Special, and 0.6" strands. In addition, the amount of shear reinforcement was varied by changing the area and spacing of stirrups. Shear reinforcement ranged from the minimum (M) steel permitted by AASHTO, to three times (3R) the amount required for the design dead and live loads.

The main variables in the test program were the percentage of shielded strands (25 and 50%), the web shear reinforcement ratio and beam end details, and the size of the prestressing strands. Details of the test program are shown in Table 2-1.

The girders were divided into three series, A,B and C. The letters A,B and C in these three series define the strand size (i.e., A,B and C represent 0.5, 0.5 special and 0.6 inch, respectively). Each series was divided into several groups as shown in Table 2-1. The shear reinforcement as well as he percentage of strand shielding were varied in each group.

The girders were generally labeled according to the strand size, degree of shielding and the amount of shear reinforcement (R, 2R, 3R, R/2 3R/2 and M) for example, A-00-R is interpreted as follows:

A	Strand size of 0.5 inch,
00	zero shielding

R Required shear reinforcement based on AASHTO Code.

Replacement of R by M indicates the minimum shear reinforcement specified in the AASHTO Code. The key to the beam designation is provided at the end of Table 2-1. The term, RD, indicates that shear reinforcement was provided in accordance with the current AASHTO Specifications (R) and confinement bars, D, were omitted.

Figures 2-1 and 2-2 show details of the test girders. Shielding patterns of strands are indicated in Figure 2-3 (a to c), with 1/2" and 1/2"S diameter strands shielded up to 5.5 ft from each end of the beam, and 0.6" diameter strands shielded up to 4.5 feet. Reinforcement details were varied at the ends of each beam as shown in Table 2-1 and Figure 2-2.

The precast beams were produced by Durastress in Leesburg; Florida. After transportation to the FDOT Structural Research Center, a top flange, 42 inches wide and 8 inches thick, was cast on all the specimens (Figure 2-1). Both girder and cast-in-place concrete slab were designed for a 28-day cylinder compressive strengths of concrete, f'_c, of 6000 psi. The initial compressive strength, f'_{ci}, at transfer was 4000 psi.

2.2 TEST SETUP AND INSTRUMENTATION

The majority of girders were tested in flexure and shear under static loads. In these tests a girder was subjected to a concentrated load applied incrementally to failure. The location of the loading varied from girder to girder, or from one end to the other, depending on the end of the beam being tested as shown in Figures 2-4 and 4-5. Upon: failure of one end of a girder, the span was adjusted to eliminate the failed zone, and another flexure or shear test was performed at the other end of the beam with the span between center to center of supports consequently reduced. It can therefore be seen, that the shear span/depth ratio was also a variable in the study.

The load was applied using a hydraulic jack with a load cell to monitor the applied load. The girders were loaded up to failure, which was defined as the inability to carry any more loads, and coincided with a complete bond slip mechanism in the majority of the tests. Upon the completion of each test the support was moved as shown in Figure 2-5 to eliminate the damaged area of the girder. The test was then repeated at the other end with the load applied at a different distance from the support

Linear voltage differential transducers (LVDT) were placed at the ends of all strands and were used to monitor the slippage in each strand continuously as the load was applied (Figure 2-6). Strain and deflection gauges were placed at different locations as shown in Figure 2-7 to collect data during testing.

TABLE 2-1 AASHTO TYPE II GIRDERS TEST PROGRAM

GIRDER NO.	SIZE OF STRANDS	NUMBER OF STRANDS	NUMBER OF SHIELDED STRANDS	% OF SHIELDING	WEB REINFORCEMENT
<u>GROUP A0</u>					
A0-00-R	0.5	16	0	00	R
A0-25-R	0.5	16	4	25	R
A0-50-R	0.5	16	8	50	R
A0-00-R(D)	0.5	16	0	00	R
<u>GROUP A1</u>					
A1-00-R	0.5	16	0	00	R
A1-00-R(D)	0.5	16	0	00	R
A1-00-R/2	0.5	16	0	00	R/2
A1-00-3R/2	0.5	16	0	00	3R/2
A1-00-M	0.5	16	0	00	M
<u>GROUP A2</u>					
A2-00-2R	0.5	16	0	00	2R
A2-00-3R	0.5	16	0	00	3R
A2-00-3R(D)	0.5	16	0	00	3R
A2-25-3R	0.5	16	4	25	3R
A2-50-3R	0.5	16	8	50	3R
<u>GROUP A3</u>					
A3-00-R(A)	0.5	16	0	00	R (WIRE MESH)
A3-00-R(B)	0.5	16	0	00	R (WIRE MESH)

TABLE 2-1 (CONT.)

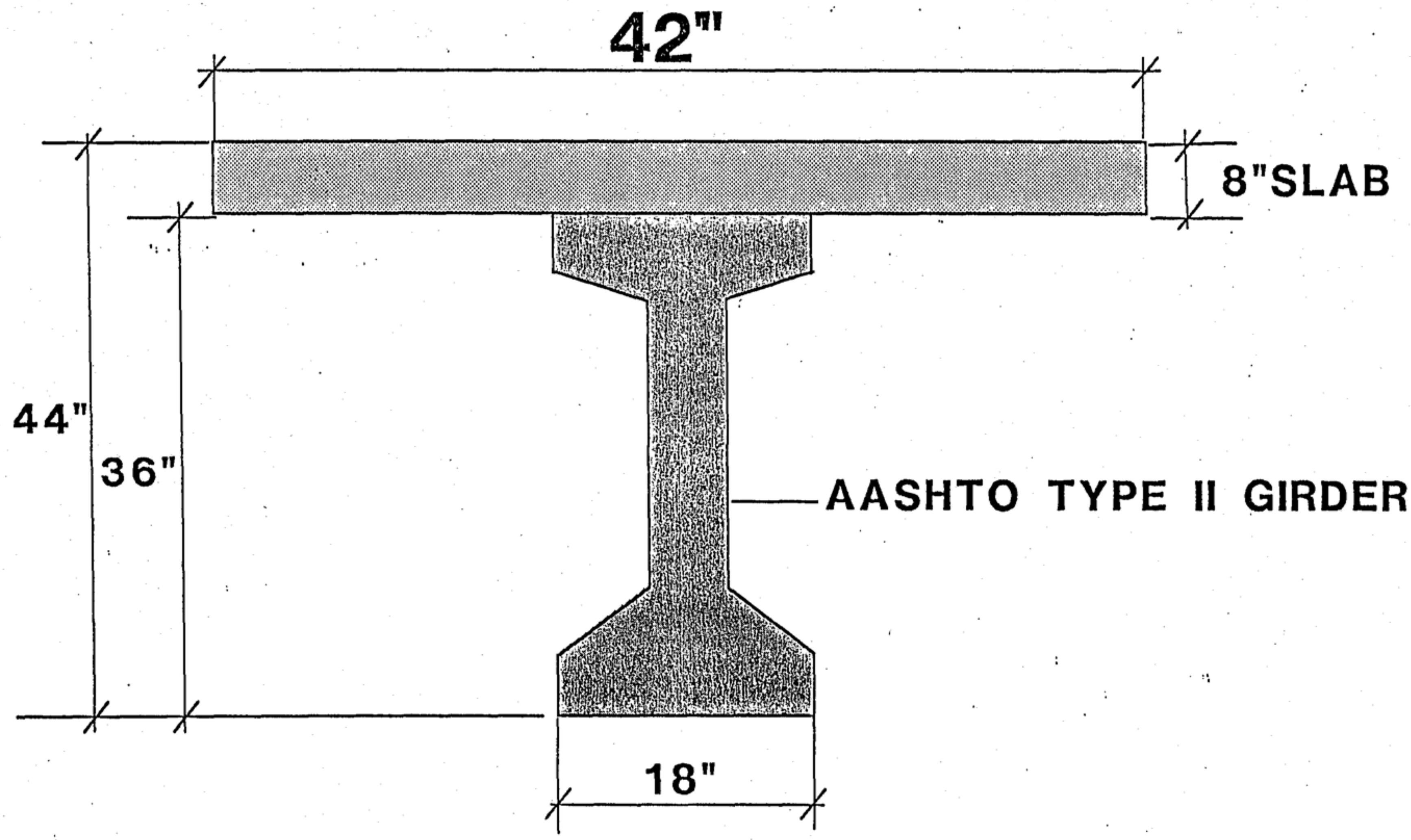
GIRDER NO.	SIZE OF STRANDS	NUMBER OF STRANDS	NUMBER OF SHIELDED STRANDS	% OF SHIELDING	WEB REINFORCEMENT
<u>GROUP B0</u>					
B0-00-R	0.5S	15	0	00	R
B0-00-2R	0.5S	15	0	00	2R
B0-00-3R	0.5S	15	0	00	3R
<u>GROUP B1</u>					
B1-00-0R	0.5S	15	0	00	0R
B1-00-R	0.5S	15	0	00	R
B1-00-2R	0.5S	15	0	00	2R
B1-00-2R(2)	0.5S	15	0	00	2R
B1-00-3R	0.5S	15	0	00	3R

TABLE 2-1 (CONT.)

GIRDER NO.	SIZE OF STRANDS	NUMBER OF STRANDS	NUMBER OF SHIELDED STRANDS	% OF SHIELDING	WEB REINFORCEMENT
GROUP C0					
C0-00-R	0.6	11	0	00	R
C0-25-R	0.6	11	3	27.3	R
C0-50-R	0.6	11	5	45.5	R
C0-00-R(D)	0.6	11	0	00	R
GROUP C1					
C1-00-R	0.6	11	0	00	R
C1-00-R(D)	0.6	11	0	00	R
C1-00-3R/2	0.6	11	0	00	3R/2
C1-25-R	0.6	11	3	27.3	R
C1-50-R	0.6	11	5	45.5	R

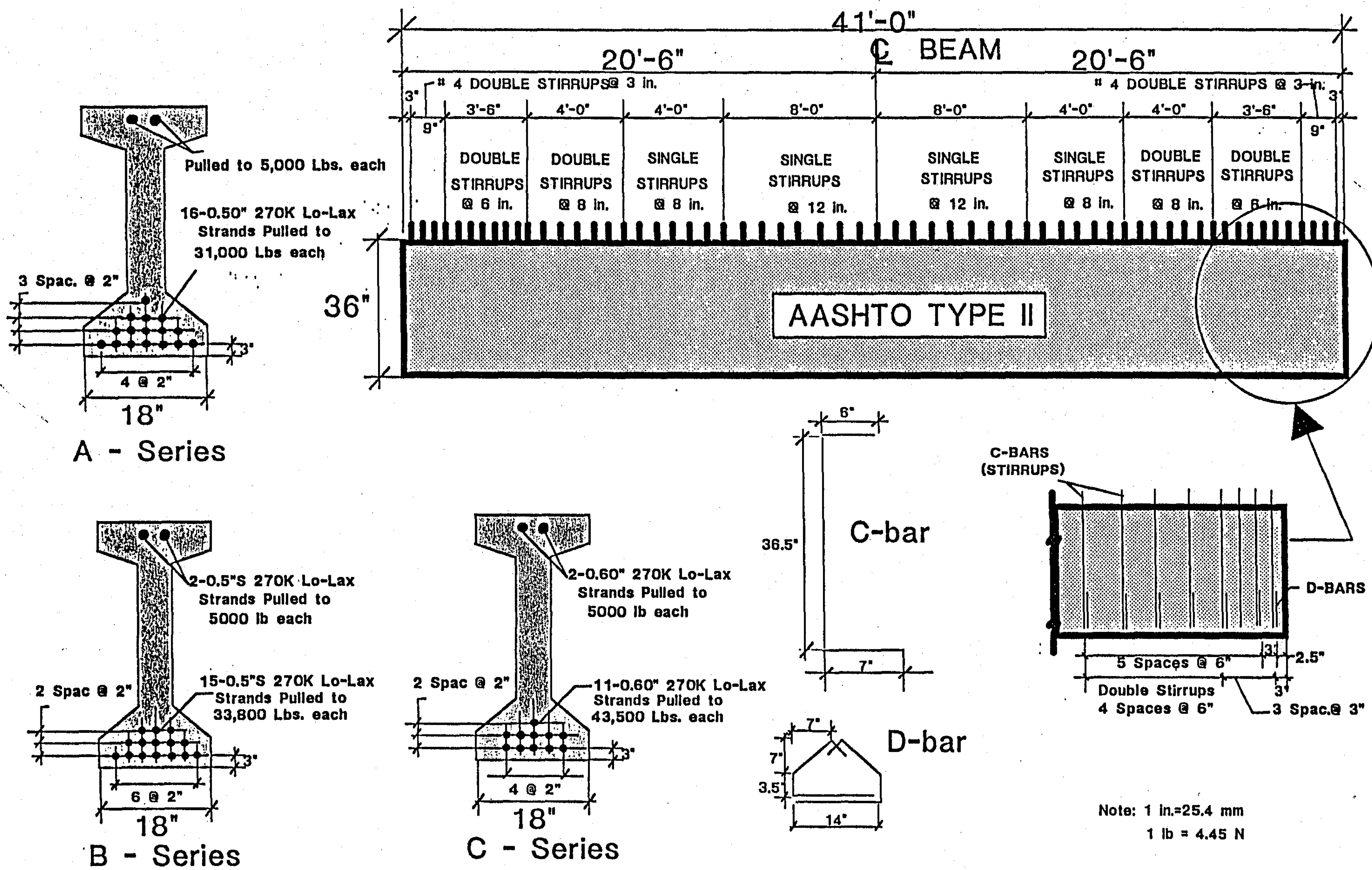
(D) : Girder has no confinement steel (bars "D") at ends
R : Required web shear reinforcement based on AASHTO Code
2R : Double the Required web shear reinforcement based on AASHTO Code
3R : Triple the Required web shear reinforcement based on AASHTO Code.
R/2 : 1/2 required web shear reinforcement.
M : Minimum web shear reinforcement.

Note: An additional Control Girder was cast and tested without top flange with a span of 20 ft.



CROSS SECTION DETAILS

FIGURE 2-1



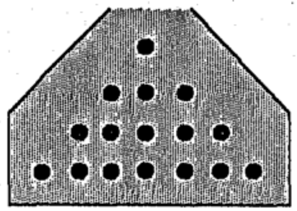
DETAILS OF TEST GIRDERS

FIGURE 2-2

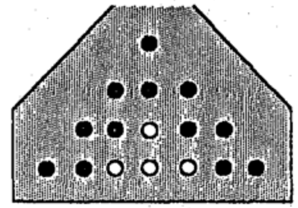
SHIELDING PATTERN FOR 1/2" DIAM. STRANDS

● UNSHIELDED STRAND

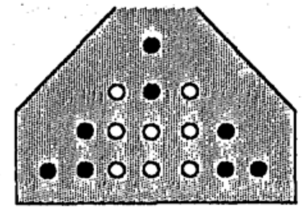
○ STRAND SHIELDED FOR 5'-6" FROM END OF BEAM



NO SHIELDED STRANDS
(0 % SHIELDING)



4 SHIELDED STRANDS
(25% SHIELDING)

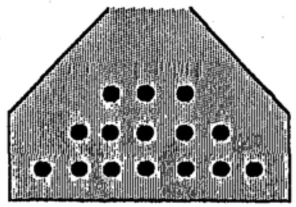


8 SHIELDED STRANDS
(50% SHIELDING)

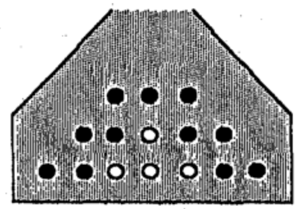
SHIELDING PATTERN FOR 1/2"S DIAM. STRANDS

● UNSHIELDED STRAND

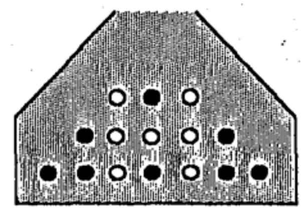
○ STRAND SHIELDED FOR 5'-6" FROM END OF BEAM



NO SHIELDED STRANDS
(0 % SHIELDING)



4 SHIELDED STRANDS
(26.7% SHIELDING)

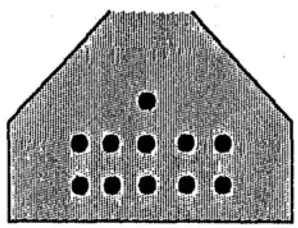


7 SHIELDED STRANDS
(46.7% SHIELDING)

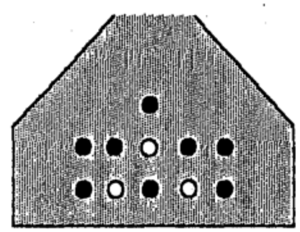
SHIELDING PATTERN FOR 0.6" DIAM. STRANDS

● UNSHIELDED STRAND

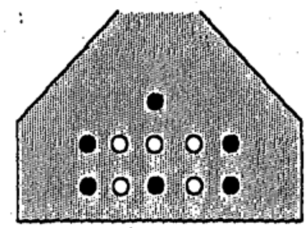
○ STRAND SHIELDED FOR 4'-6" FROM END OF BEAM



NO SHIELDED STRANDS
(0 % SHIELDING)



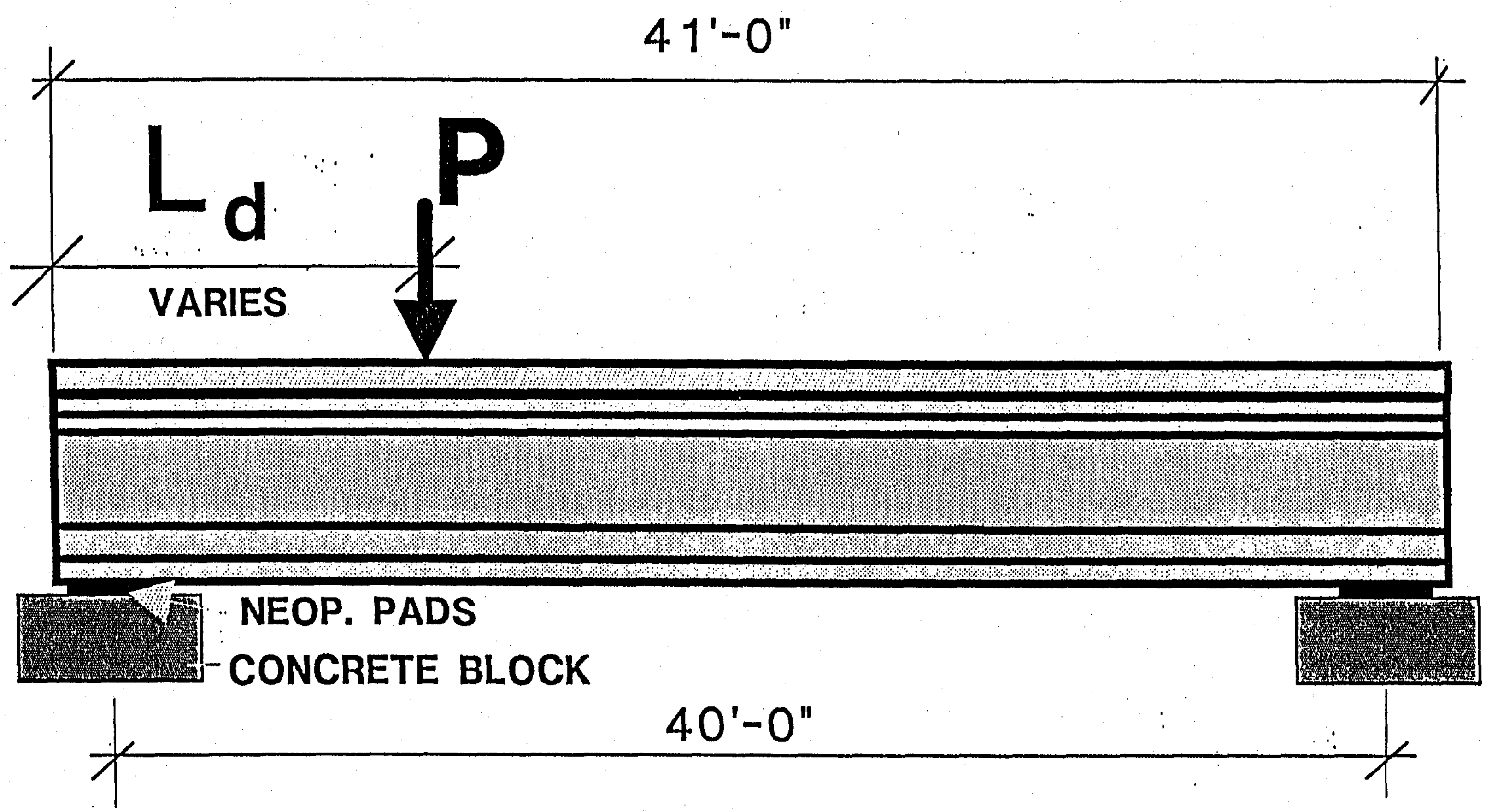
3 SHIELDED STRANDS
(27% SHIELDING)



5 SHIELDED STRANDS
(45% SHIELDING)

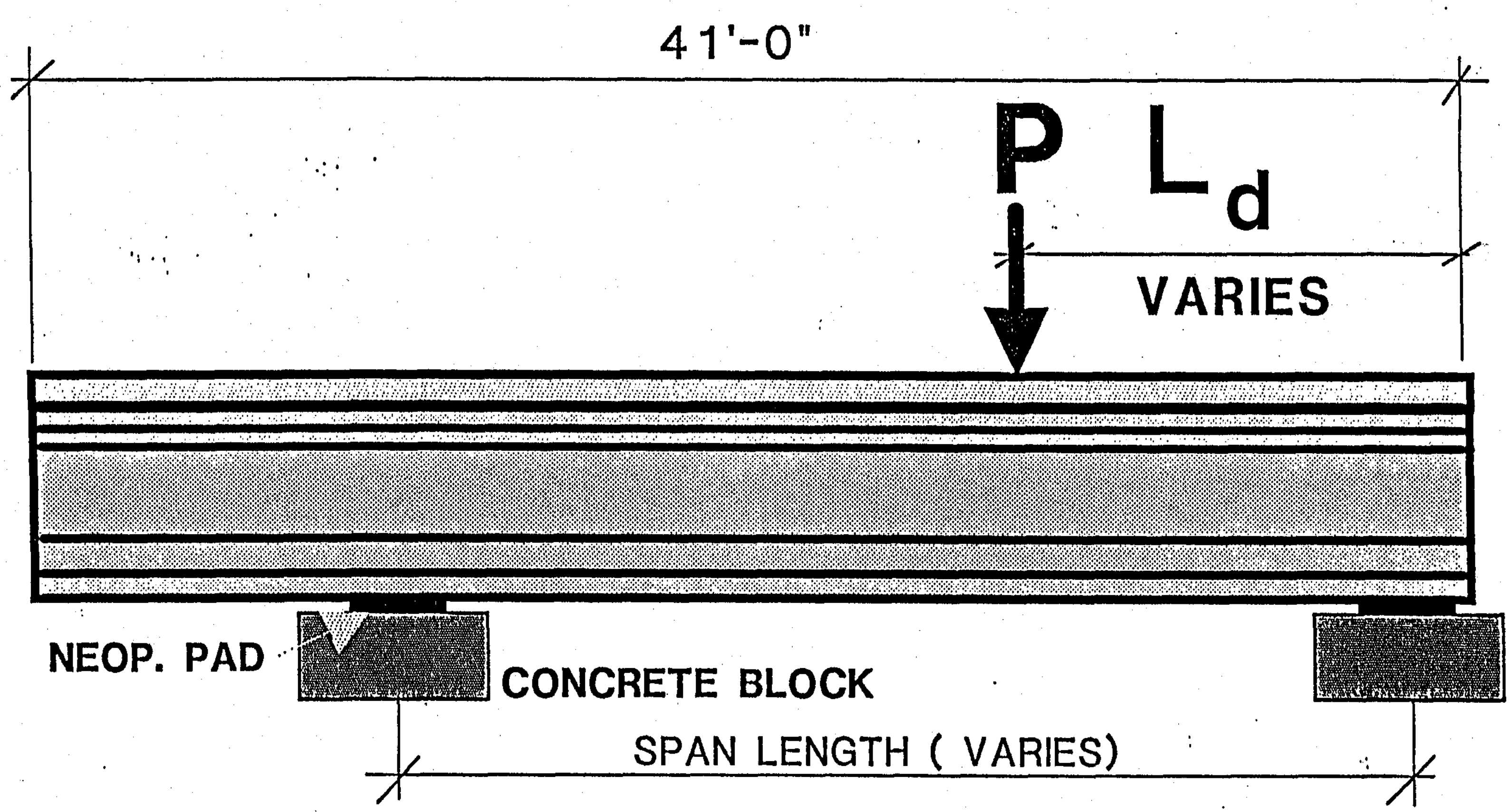
DETAILS OF SHIELDING

FIGURE 2-3



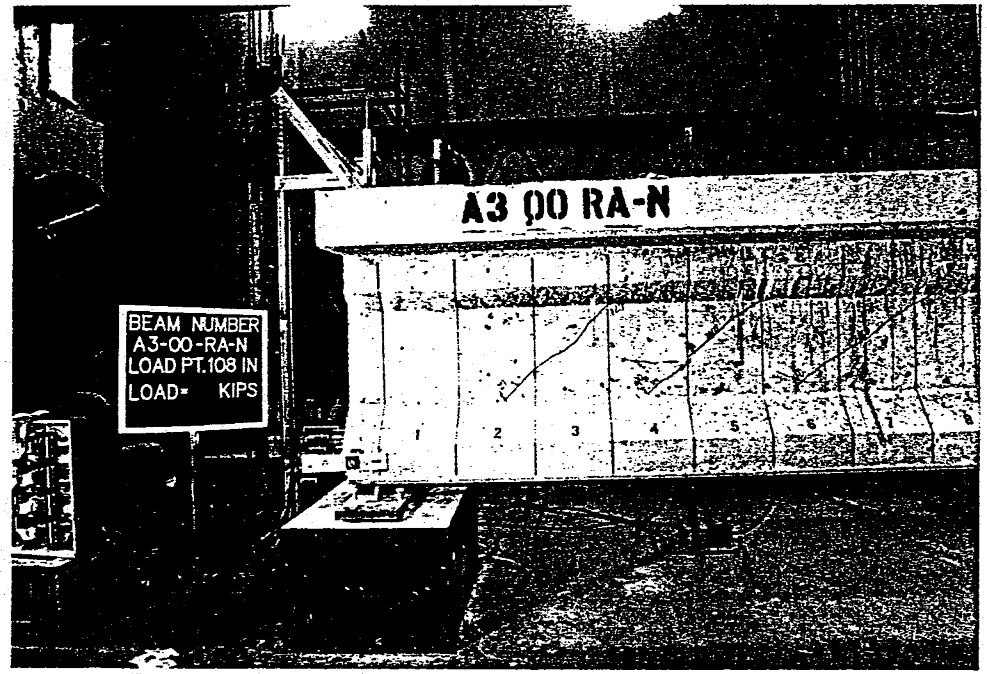
TEST SETUP - NORTH END

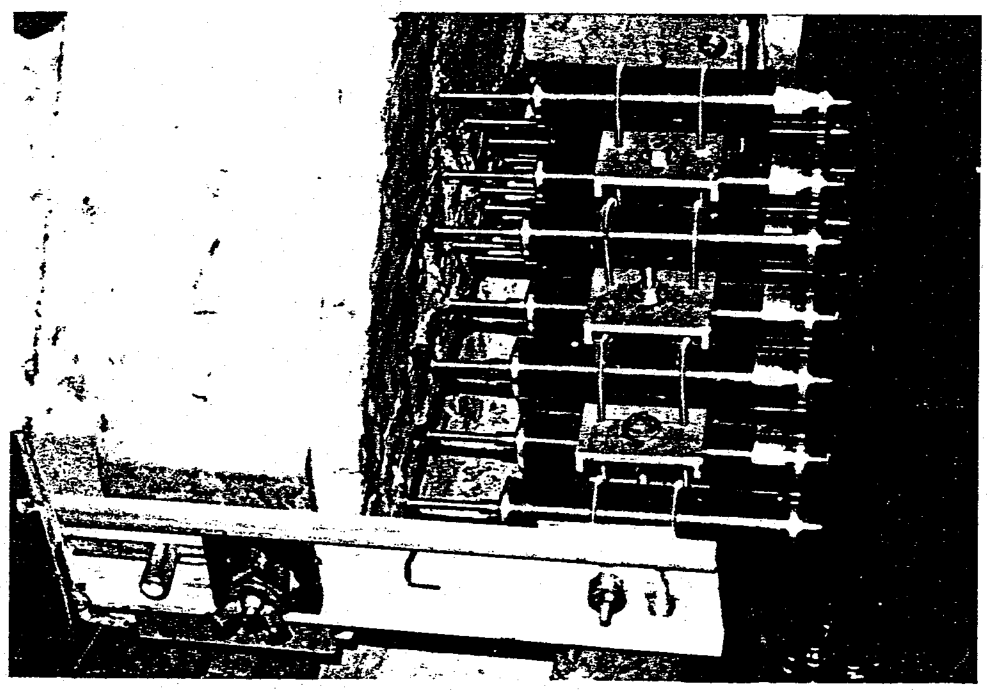
FIGURE 2-4



TEST SETUP - SOUTH END

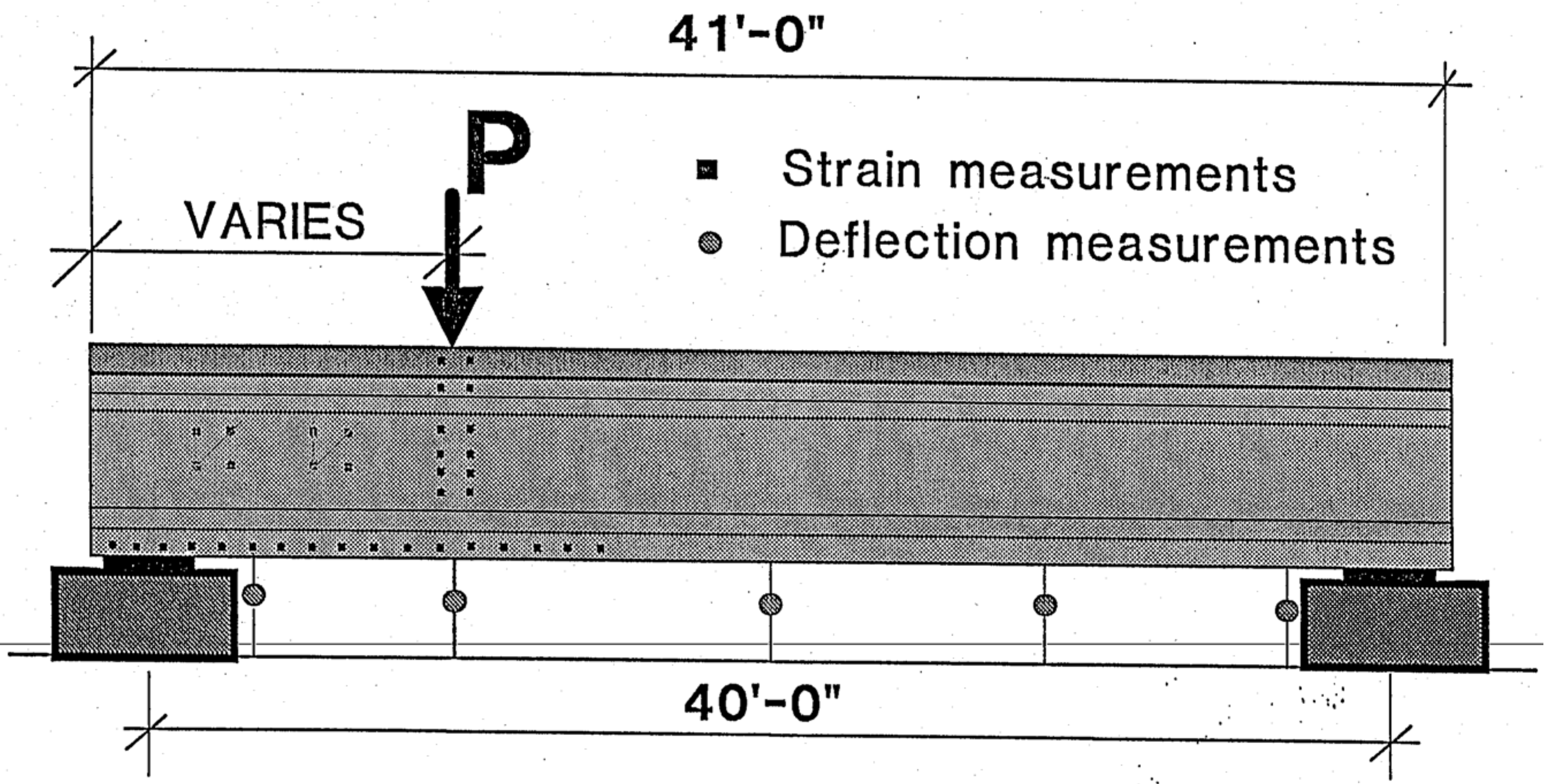
FIGURE 2-5





TEST SETUP : LVDT'S FOR STRAND SLIPPAGE

Figure 2-6



STRAIN AND DEFLECTION MEASUREMENTS

FIGURE 2- 7

CHAPTER 3
ANALYSIS AND DESIGN OF GIRDERS

The test girders were analyzed using the FDOT Prestressed Concrete Bridge Girder Design Program (PCBGD). This program is used in the design or analysis of simply supported prestressed concrete :girders. The results of the review mode analysis were used to determine the applied shear forces due to dead loads (composite and non composite) and live load (HS-20 truck).

3.1 FDOT PRESTRESSED CONCRETE BRIDGE GIRDER DESIGN PROGRAM (PCBGD)

A typical input for the PCBGD program is shown in Table 3-1 for Girder Al-00-R. These input values are for a 40 ft. simply supported girder, with girders spaced at 10 ft., and an 8 in. concrete slab. The live load distribution factor, S/5.5, specified in Table 3.23.1 of the current AASHTO Code is divided by a factor of 2.0 since the girder program considers axle loads instead of wheel loads. This results in a distribution factor S/11.0 = 0.91 for the test girders. The transformed slab width is input as 42 in. corresponding to the actual slab width of the test girders, as opposed to the effective slab width in the hypothetical bridge. This reduced slab width does not effect the shear forces resulting from the beam program analysis, which includes the entire slab width (10 ft.) in the load computations. However, this reduction slightly affects the computed shear capacity provided by the concrete (V_{ci} or V_{cw}).

The input for the prestressed beam program also includes non composite dead loads, material properties, and section properties. Section properties were computed based on the geometry and dimensions of the girder and slab. The material properties shown in the input data include the unit weight of concrete, the modules of elasticity of concrete, the modules of elasticity of prestressing steel, strand clearances, and the allowable tensile and compressive stresses of concrete.

Non composite dead loads consists of the weights of the girder, the slab and the stay-in-place forms. The weight of the latter is computed by subtracting the width of. the beam top from the girder spacing (c/c) and then multiplying this distance by the weight of the forms.

In the non composite dead load calculations, build-up over the girder is ignored, and the program automatically computes the dead load of the girder and slab. The composite dead load consists of the weight of two concrete barriers (418 lb/ft each) and future asphalt surfacing (15 psf). This total load was distributed equally over four girders.

3.1.1 PCBGD Program Output

The output from the program is shown in Appendix Al for girder Al-00-R. This output includes the input data, section properties, prestressed beam design (number, size, pattern and force of strands), beam deflection and camber, summaries of applied moments and shear forces along the span, girder stresses, shear strength V_c, for the concrete and stirrup design (bar size and spacing). The calculations in the prestressed girder program are based on the provisions of the current AASHTO Code.

3.2 SHEAR CALCULATIONS USING LOTUS SPREADSHEET

A Lotus spreadsheet was written and used in all the shear calculations. This spreadsheet performs all the required calculations, except those performed by the Prestressed Girder program (PCBGD) and the RESPONSE Program (Ref. 5) as follows:

1) computes the composite and non composite dead loads needed for input in the prestressed beam program;
2) interpolates values of moment and shear, factored shear and moment values, and required shear and moment capacities, V_u/φ and M_u/φ;
3) interpolates V_c values and computes shear capacities, V_n and V_s, based on the current AASHTO Code;

4) computes the shear capacity, V_n, V_c and V_s, and related factors (v_c, ε_x, and θ) in accordance with the proposed Code[4];

5) computes the shear capacities as stated in item (4) above but assuming a constant value (0.003) for ε_x;

6) computes input values for the RESPONSE program; and

7) computes the dead, live and total shear forces and moments along the girder due to the combination of dead load and the applied test load.

The spreadsheet results for Beam Al-00-R. are shown in Appendix A2. As an illustration of the spreadsheet calculations, the relevant values are computed at 0.3L for Girder Al-00-R in the following section. .

3.3 CALCULATIONS FOR GIRDER Al-00-R AT 0.3 L

3.3.1 Vn Required

From the shear forces computed by PCBGD program, the required shear capacity, V_n REQUIRED, was determined by dividing the factored shear forces by the strength reduction factor, $\varphi = 0.90$ for the current AASHTO Code, and $\varphi = 0.85$ for the proposed code, as follows:

$$Vn = V_u/\varphi = (1.3\ V_D + 2.17\ V_{(L+1)})/\varphi \qquad [1]$$

At 0.3L, of Girder Al-00-R, the shear forces due to dead load, V_d, and live load, V_L, are 15.4K and 41.7K, respectively. Thus, the factored shear, V_u, is computed as:

$$Vu = 1.3(V_D + 5/3(V_{(L+I)}) = 110.4K.$$

The shear strength required is computed by dividing the strength reduction factor. For the current AASHTO Code; $V_n = V_u/0.9 = 122.6K$.

For the proposed AASHTO Code; $V_n = V_u/0.85 = 129.8K$.

3.3.2 V_n (Current AASHTO)

The PCBGD program computes the values of shear strength provided by the concrete, V_{ci} or V_{cw}, at the tenth points of the span. In accordance with the AASHTO code, the smaller of the two values was used as the shear strength of the concrete, V_c. Values of V_c at other intermediate points were found by linear interpolation using the values of V_c at the tenth points. The total shear capacity or strength provided, V_n, was computed as:

$$V_n = V_c + V_s \qquad [2]$$

Where, V_s is the shear strength provided by shear reinforcement and is calculated using current AASHTO provisions.

$$V_s = A_v f_y d/s \qquad [3]$$

Where, A_v = area of shear reinforcement bars of diameter, d, having a yield strength, fy, and spaced at a distance, s, c/c.

The location of the shear reinforcement for beam Al-00-R is shown in Figure 2-2. At locations where the stirrup spacing or the total stirrup area changes, average values of s and A_v values are used in Eqn. [3]. For example, the shear strength provided by the steel V_s, in girder Al-00-R at 0.3L is calculated as:

$$Vs = (0.2)(60)(40.25)1(10) = 48.3 \text{ Kips}.$$

At 0.3L of Girder A1-00-R, the shear capacity provided by the concrete is 95.6 kips (the smaller value of V_{ci}=95.6 K and V_{cw}=166.6 K).

Thus, total shear capacity at 0.3L = 95.6 + 48.3 = 143.3 Kips.

3.3.3 Vn (Proposed AASHTO)

The shear capacity, Vn, was calculated in accordance with Section 5.8 of the proposed AASHTO Code (Ref. 4), which gives the nominal shear resistance, V_n, at a section as:

$$V_n = V_c + V_s + V_p \qquad [4]$$

Where V_p is the shear strength provided by of prestressing tendons which is equal to zero in girders with straight tendons. An upper limit for V_n is given as:

$$V_n \leq 0.25f'_c b_v d_v + V_p \qquad [5]$$

For the test girders, the shear resistance would be limited to 349 kips. The shear resistance provided by the concrete and the stirrups (a = 90) are, respectively, given in Eqns. (6) and (7):

$$V_c = \beta \sqrt{f'_c}\, b_v d_v \qquad [6]$$

$$V_s = A_v f_y d_v \cot\theta / s \qquad [7]$$

In Eqns. (5)-(7), d_v is the effective shear depth calculated as the distance between the resultants of the tensile and compressive forces due to flexure. Thus, the value of d_v varies only in the end region for beams with shielded strands. The effective web width, b_v, is 6 in. for AASHTO type II girders.

In the proposed code two tables are provided for the determination of β and θ; one table for sections with transverse reinforcement, and a second table for sections without transverse reinforcement. These Tables are reproduced as Table 3-2(a and b) in this report. All of the test girders, except B1-00-OR, contained transverse shear reinforcement.

The tabulated values in Table 3-2 are based upon the assumption that the diagonal cracks have a 12 in. spacing. In order to use this table, it is necessary to compute two values, the applied shear stress, v, and the longitudinal strain at mid-depth of the member, ε_x. These values are computed in accordance with the proposed code provisions using Equations (8) and (9), respectively:

$$v = (V_u - \varphi Vp)/ (\varphi\, b_v\, d_v) \qquad [8]$$

$$\varepsilon_x = [(M_u/d_v)+(0.5N_u)+(0.5V_u \cot\theta) -(A_{ps}f_{se})]/(E_sA_s + E_pA_{ps}) \qquad [9]$$

The terms in the above two standard equations are defined in the list of notations. Once these values are computed the table may be entered with values of v/f'c and (ε_x x1000) to determine the values of constants β and θ. It is necessary to estimate initially the inclination of diagonal cracks, θ in order to calculate the value of ε_x. The code recommends a value of 30 degrees as a starting point in the iteration process for calculating ε_x. The computations for the shear capacity were made at points along the beam and near the support by applying the equations for strain and shear stress given in Eqns. (8) and (9), respectively. In the strain equation the values for $Mu\backslash_u$ and V_u were computed using the PCBGD program.

The Lotus Spreadsheet was used to make the necessary calculations. The spreadsheet uses Equations (4), (6) and (7) to compute the shear capacity after the values of β, ε_x, d_v, v/f'_c, and f_{se} are determined. The effective prestress force in the strands was taken from PCBGD, which calculates the jacking force for the 1/2 in. dia., 270 ksi prestressing strands to be 30,990 lbs, and the prestress loss as 24.02 percent. Thus, the effective prestress was computed

$$f_{se} = Pi\ (\ 1 - 0.2402)\ /\ A_P = 30{,}990\ (1\text{-}0.2402)/\ 0.153 = 154\ \text{Ksi}$$

The effective shear depth, d_v, was computed as

$$d_v = jd = (d - a/2).$$

In the above calculations, d is the effective depth of the section (beam plus slab) and a is the depth of the equivalent rectangular stress block As shown in Figure 2-1, the total height of the section is 44 in. and the strand pattern is such that 7, 5, 3 and 1 strands, are located at 2, 4, 6 and 8 inches, respectively, from the. bottom of the beam. Thus, the center of gravity of the prestressing strands is 3.75 in. from the bottom of the girder. Therefore, d = 44.0 - 3.75 = 40.25 in.

The value of a is computed after determination of the stress, f^*_{su}, in the prestressing steel at ultimate as follows:

$$f^*_{su} = f'_s\,[1-(\gamma^*/\beta 1)(\rho^*\, f'_s / f'_c]$$

$$= (270)[1-(0.28/0.75)(0.001448)(270/6)] = 263.4 \text{ ksi.}$$

$$a \quad = (A^*_s\, f^*_{su})/(0.85\, f'_c b)$$

$$a \quad = (2.448)(263.43)/[(0.85)(6)(42)] = 3.01 \text{ in.}$$

$$d_v \quad = jd = (d\text{-}a/2) = 40.25 - (3.01/2) = 38.74 \text{ in.}$$

The terms in the last two standard equations are defined in the list of notations.

The calculation of shear capacity at 0.3L for Beam A1-00-R, using the provisions of the proposed code is described in the following paragraph.

From the FDOT's PCBGD program, the values of the factored shear and moment are V_u=110.37 K and M_u= 1504.14 K-ft. In the study, both the factored axial force, N_u, and the vertical prestress force, V_p, were zero. The value of v/f'_c is then computed as:

$$v = (V_u - \varphi\, V_p)/(\varphi\, b_v d_v)$$

$$= (110.37)/(0.90(6)(38.74)) = 0.559 \text{ ksi}$$

$$V/f'c = (0.559)/(6) = 0.093$$

The longitudinal strain, ε_x, is computed from Eqn. (9):

$$\varepsilon_x = [(1504.14)(12)/38.74)+(0)+(0.5(110.37)(\cot 30^\circ) -(2.448)(154]/(0+(28,000)(2.448)$$

$$= 0.0026918$$

ε_x x (1000)=2.6918.

Applying these values for ε_x and v/f'_c in Table 3(a), results in $\theta = 38^\circ$ and $\beta = 1.00$. The shear capacity is then computed substituting $A_v = 0.2$ in^2, and s =10.00 in. as follows:

$$Vc = 1.00\,(\sqrt{6000})\,(6)(38.74) = 18.0$$

Kips From Eqn. (7):

$$V_s = (0.2)(60)(38.74)(\cot 38^\circ)/10 = 59.5 \text{ Kips}$$

$$V_n = V_c + V_s = 18.0 + 59.5 = 77.5 \text{ Kips}$$

The shear capacity values computed at other locations along the beam are shown on page four (4) of Appendix A2.

3.3.4 V_n PROPOSED (x =0.003)

As a simplification, the proposed code allows the shear capacity to be computed assuming a value of $\varepsilon_x = 0.003$. The shear capacities at different sections along the girder are computed in the same way as those for the proposed AASHTO Code (as described in the preceding section) with the exception that longitudinal strain is assumed equal to 0.003. Thus, for use in Table 3-2, ε_x (1000) = 3.00. In this particular example, at 0.3L of Beam Al-00-R, both the values 2.69 and 3.00 result in the use of the column for $\varepsilon_x = 3.00$ of the table. Therefore, the values calculated for the shear capacities are the same as those using the computed value for Ex as discussed in the previous section: $\theta = 38^\circ$, $\beta = 1.00$, $V_c = 18.0$ K, V_S=59.5K and V_n=77.5 K.

3.3.5 V_n (RESPONSE)

The computer program RESPONSE is based on the formulation presented in Reference (5) which provides details of the formulation for the Modified Compression Field Theory (MCFT) and the pertinent equations used in the program. The maximum shear capacity given by RESPONSE is taken as the shear strength, V_n RESPONSE. Using MCFT, this program computes the load-deformation response of a member using equilibrium, compatibility, and stress-strain relationships. It is pointed out in Reference (5) that for a given level of shear, V, there are five unknowns: the stress in the longitudinal bars, G; the stress in the longitudinal prestressing tendons, f_p; the stress in the stirrups, f_v; the diagonal compressive stress in the concrete, f2; and the inclination, 0 , of the diagonal compressive stresses. While the original Compression Field Theory ignores the tensile stresses in the concrete, the MCFT includes the effects of diagonal tensile stresses, f$, in the shear resistance.

As in the current AASHTO provisions, the shear resistance of a member is given as the sum of contributions from concrete and shear reinforcement:

$$V = f_1 \, b_w \, jd \cot\theta + A_v \, f_v \, jd \cot\theta / s \qquad [10]$$

The quantity, f_v, is the average tensile stress in the shear reinforcement.

Input values for RESPONSE are calculated using the Lotus spreadsheet. The program input and output for test girder A1-00-R at 0.3L are shown in Appendix A3. Although three concrete types are shown in the input, only type 1 is used. With f'_c = 6000 psi, and using a maximum compressive strain in the concrete of 3.00 millistrain, the program selects a value of concrete modules of elasticity, E_c = 4000 ksi. This is approximately the value recommended by the FDOT's Structures Design Guidelines[6] and used in the PCBGD program.

The tensile strength of the concrete, f_{cr}, is assumed to be 4 $\sqrt{f'_c}$ = 310 psi. A tension stiffening factor of 0.49 was chosen for bonded strands in sustained and repeated loading. For low relaxation strands, strain hardening is assumed to begin at 10 milli-strain with rupture occurring at 40 milli-strain.

The stress-strain response of the prestressing strands is represented by a modified Ramberg-Osgood function. The constants A, B, and C in the input data are the factors corresponding to low relaxation strands having an ultimate tensile stress of 270 ksi.

The section properties computed for the test beam are shown in the input data of Appendix A3. The distance to the moment axis is the distance to the neutral axis measured from the bottom of the beam. The shear depth, jd, is the same as the shear depth, d_v, computed for the PROPOSED AASHTO result The distance to the web strain, ex, and the distance to the center of the web are computed as 23.12 in., corresponding to a distance jd/2 above the centroid of the flexural reinforcement

The equations used to compute the average crack spacings, s_{mx} and s_{mv} for a member subject to longitudinal tension and transverse tension, respectively, are given as follows:

$$s_{mx} = 2(c_x + s_x /10) + 0.25k_1\, d_{bx} /\rho_x \qquad [11]$$

$$s_{mv} = 2(c_v + s/10) + 0.25k_1\, d_{bv} /\rho_v \qquad [12]$$

Where;

$$\rho_v = A_v/(b_w\, s) \qquad [13]$$

$$\rho_x = (A_{sx} + A_{px})/A_c \qquad [14]$$

In the above equations, ρ_v is computed from the shear reinforcement, and k_l = 0.4 for deformed bars and 0.8 for bonded strands. Similarly, ρ_x is computed from the longitudinal reinforcement. The first term, A_{sx}, in Eqn. (14) applies to reinforcing steel and the second term, A_{px}, applies to prestressed steel, while Ac is the area of the concrete section. In Eqns (11) and (12), the distance c is the maximum distance from the reinforcement.

For test girder Al-00-R, both 1/2 in. (#4) stirrups and 1/2 in. diameter prestressing strands are used. Thus:

s_{mx} = 2(12.5+ (2/10)) + 0.25(0.8)(05)/(2.754/705)) = 51.0 in.

s_{mv} = 2(3.5 + (10/10)) + 0.25(0.4)(0.5)/(0.2/(6)(10)) = 24.0 in.

The area of the longitudinal steel (2.754 in^2), includes the two prestressing strands in the top flange which are stressed to 5000 lbs. For double stirrups the maximum distance away from the steel is 2.0 inches; for single stirrups the maximum distance away from the steel is 3.5 inches; giving an average of 2.75 inches. The maximum aggregate size is 0.75 in. which is consistent with class III concrete in accordance with Ref. (7).

The concrete layers and tendon layers are input to correspond to the beam section dimensions and the prestressing. steel layout (Figure 2-1). The bottom cover for the prestressing strands is 2 in. In general, for the various test girders, the RESPONSE input data is altered as appropriate to represent the test girder properties (strand and stirrup data).

With the appropriate input, the program RESPONSE is run for each girder at the location where the shear capacity is desired. The fact that stirrup details change along the length of the girder is taken into account. Also, strand shielding where present, is taken into consideration. When the program is run, the following options are selected: SHEAR AND AXIAL LOAD, CONSTANT AXIAL LOAD, FULL RESPONSE. The axial force is taken as zero, and a parabolic concrete model is used for the concrete stress-strain relationship. The RESPONSE program output for girder Al-00-R is shown in Appendix A3. The output includes axial loads, shears, moments, strains, and angles of diagonal cracks. From this output data, the maximum shear of 144.9 kips, was selected.

For each test girder the input and output for Response were obtained at the tenth points, at points near the support, and several other intermediate locations. The maximum shear value corresponding to zero axial load is selected as the Vn RESPONSE value, and is plotted for comparison with the other shear capacities.

TABLE 3-1. INPUT FOR GIRDER A1-00-R

FLORIDA DEPARTMENT OF TRANSPORTATION

PRESTRESSED CONCRETE BRIDGE GIRDER DESIGN PROGRAM - VERSION 2.3

*TEST BEAM REVIEW: A1-00-R TYPE II BEAM SPAN= 40.0 FT
*B. ROBINSON 488-6179
*

***** REVIEW ONLY ***** REVIEW ONLY ***** REVIEW ONLY ***** REVIEW ONLY ***** REVIEW ONLY ***** REVIEW ONLY *****

SSSSSSSSSSSSSSSSSSSSSSSSSSSSSSSSSSSSSS S T R A I G H T S T R A N D S SSSSSSSSSSSSSSSSSSSSSSSSSSSSSSSSSSSSSS

****** L O W - R E L A X A T I O N ******

*** INPUT DATA ***

BEAM TYPE	= 2	UNIT WT. BEAM CONC.	= 150. PCF	STRAND SIZE	= 1/2 IN
SPAN LENGTH	= 40.00 FT	UNIT WT. SLAB CONC.	= 150. PCF	STRAND ULT STRENGTH	= 270 K
BEAM SPACING	= 10.00 FT	28-DAY ST.(SLAB CONC.)	= 6000. PSI	NO. OF WEB STRANDS	= 1
SLAB THICKNESS	= 8.00 IN	E(BM.CONC.)	= 4.00 E(06)PSI	GRID SIZE	= 2.00 IN
L.L. DIST. FACTOR	= 0.91	E(SLAB CONC.)	= 4.00 E(06)PSI	STRAND CL. BOTT. BEAM	= 2.00 IN
TRANS. SLAB WIDTH	= 42.00 IN	E(PRESTRESS STEEL)	= 28.00 E(06)PSI	STRAND CL. SIDE BEAM	= 3.00 IN
UNIF. D.L. N-COMP	= 0.180 KLF	AASHTO L.L. + DIAF. WT.	= HS-20 (+5%)	MAX COMP BM. CONC(ALLOW)	= 2400. PSI
UNIF. D.L. COMP	= 0.359 KLF	RAILROAD L.L.	= E- 0.	MAX. TENSION AT DES. LOAD	= -232. PSI
DIAPHRAGM	= 5% L.L.			COPE DIMENSION	= 0.00 IN

*** SECTION PROPERTIES ***

		PRECAST	COMPOSITE
AREA	=	369.00	705.00 IN2
WEIGHT	=	384.37	1384.37 PLF
MOMENT OF INERTIA	=	50979.00	155508.70 IN4
YB BEAM	=	15.83	27.35 IN
SECTION MODULUS BOTTOM	=	3220.40	5686.02 IN3
YT BEAM	=	20.17	8.65 IN
SECTION MODULUS TOP	=	2527.47	17976.47 IN3
YTS SLAB	=		16.65 IN
SECTION MODULUS SLAB	=		9339.48 IN3
HEIGHT	=	36.00	44.00 IN

*** BEAM DIMENSIONS (INCHES) ***

B= 18.00 W= 6.00 C= 6.00 E= 6.00 A= 12.00 H= 6.00 G= 3.00 Q= 0.00 R= 0.00

*** CONCENTRATED LOADS APPLIED TO NON-COMPOSITE SECTION ***

LOAD (KIPS)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
DIST. FROM LT. REACT. (FT)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

*** CONCENTRATED STATIC LOADS APPLIED TO COMPOSITE SECTION ***

LOAD (KIPS)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
DIST. FROM LT. REACT. (FT)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

*** CONCENTRATED LIVE LOADS APPLIED TO COMPOSITE SECTION ***

LOAD (KIPS)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
DIST. FROM LT. LOAD (FT)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Table 3-2. Values of Theta and Beta

(a) - Values of θ and β for Sections with Transverse Reinforcement

$\frac{v}{f'_c}$		$\epsilon_x \times 1{,}000$ ≤ 0	≤ 0.25	≤ 0.50	≤ 1.00	≤ 1.50	≤ 2.00	≤ 3.00
≤ 0.050	θ	28°	31°	34°	38°	41°	43°	45°
	β	5.24	3.70	3.01	2.33	1.95	1.72	1.39
≤ 0.075	θ	28°	30°	30°	36°	40°	42°	43°
	β	4.86	3.37	2.48	2.15	1.90	1.65	1.25
≤ 0.100	θ	22°	26°	30°	36°	38°	38°	38°
	β	2.71	2.42	2.31	2.08	1.72	1.39	1.00
≤ 0.150	θ	25°	28°	31°	34°	34°	34°	35°
	β	2.53	2.25	2.13	1.73	1.30	1.04	0.77
≤ 0.200	θ	27°	30°	33°	34°	34°	37°	41°
	β	2.16	2.13	2.09	1.52	1.08	1.11	0.99
≤ 0.250	θ	30°	32°	34°	36°	39°	42°	49°
	β	2.25	2.00	1.87	1.45	1.37	1.32	1.24

(b) - Values of θ and β for Sections without Transverse Reinforcement

d		$\epsilon_x \times 1{,}000$ ≤ 0	≤ 0.25	≤ 0.50	≤ 1.00	≤ 1.50	≤ 2.00	≤ 3.00
$\leq$ 10 in.	θ	30°	34°	37°	41°	43°	45°	48°
	β	4.65	3.40	2.83	2.21	1.87	1.64	1.35
$\leq$ 15 in.	θ	32°	37°	41°	45°	48°	50°	53°
	β	4.47	3.15	2.59	1.99	1.67	1.45	1.17
$\leq$ 25 in.	θ	35°	42°	46°	51°	54°	57°	61°
	β	4.24	2.82	2.27	1.70	1.39	1.19	0.94
$\leq$ 50 in.	θ	38°	48°	53°	60°	64°	66°	70°
	β	3.90	2.39	1.82	1.28	1.01	0.84	0.63
$\leq$ 100 in.	θ	42°	56°	62°	69°	73°	75°	78°
	β	3.55	1.87	1.35	0.88	0.65	0.52	0.37

(Source: Proposed Revision to the AASHTO code, Tables 5.8.3.4.2-1 and 5.8.3.4.2-2, pages 5-70 and 5-71, June 4, 1991.)

CHAPTER 4

PRESENTATION AND DISCUSSION OF RESULTS

4.1 GENERAL

Both plots and tables showing the predicted shear capacities and the test shear values are used to summarize the results. Table 4-1 summarizes the results of the test program, and includes measured and predicted shear capacities, the shear spans, and the ratios of test capacity to predicted capacity. Appendix A4 presents plots of predicted and test shear capacities of all test girders,. while Appendix A5 contains crack pattern charts.

It should be noted that for those beams failing in flexure, the actual test values of the applied shear force, V, are used in calculating the shear ratios for the different approaches. As can be seen from this table, for those beams failing in shear, the best predictions of shear capacities are obtained applying the current AASHTO code followed by the RESPONSE program. Table 4-2 shows the means, coefficients of variation, and standard deviations for the shear capacity ratio for the girders which failed in shear. As shown in Table 4-2, for all Group A girders failing in shear the mean and standard deviation of the shear capacity ratios are 1.17 and 0.15, respectively, for the current AASHTO Code.

Figure 4-1 shows the results for beam AO-00-R. The shear capacities are plotted at the twentieth points along the span. Several of the trends in this figure are typical of the results shown in Appendix A4. As shown in Figure 4-1, the proposed code predicts elevated shear strengths near the. support and greatly reduced shear strengths beyond the 0.2 L . When the longitudinal strain is assumed to be ε_x=0.003, instead of the computed value, the proposed code predicts very low shear strengths near the support and is very conservative. The values computed by the two methods of the proposed code diverge towards the supports, but converge as the distance from the support increases and become the same from approximately 0.25L to mid span.

At distances near 0.2 L and greater, the values from the proposed code values are lower than the shear capacity values predicted by the current AASHTO code, although the latter values are greater than those required ($V_u/0.85$). For both plots for the proposed code, the values at h/2 are assumed equal to the values at 43 in. since the proposed code states that sections less than a distance d_v from the face of the support, may be designed for the same factored shear, V_u, as that computed at a distance d_v. This is due to the fact that the loading produces compression in the end of the girder.

Near the ends of the girder, the shear strengths predicted by the current AASHTO Code provisions lie between the extreme values for the proposed code. At h/2 and at d_v (38.7in.), the current AASHTO values are approximately equal to the proposed code values computed by assuming a longitudinal strain is equal to $\varepsilon_x = 0.003$. Over the entire length of the girder the shear capacity based on the current AASHTO provisions remains greater than the proposed code values with $\varepsilon_x = 0.003$. In Fig. 4-1, the test shear strengths are greater than all the predicted values.

In general, the form of the shear capacity plots shown in Figure 4-1 for beam AO-00-R is typical of plots for all other girders except those with shielded strands, minimum shear reinforcement and two to three times the amount of shear steel required by the current AASHTO Code. Shear capacity plots for all beams are shown in Appendix A4.

Figure 4-2 shows the shear capacity plots for girder Al-00-M which was provided with the minimum shear reinforcement permitted by the current AASHTO Specifications. It is seen that for this girder the shear capacities predicted by the proposed code are lower than those predicted by the current AASHTO code, which gives the best prediction of shear strengths.

For girders with shielded strands, such as beam AO-25-R (Figure 4-3), the calculated shear capacity resulting from computing the longitudinal strain, ex, is greatly reduced at the end region of the girder. These values become approximately equal to or less than the current

AASHTO predicted shear capacities. Therefore, both the proposed and current codes account for the reduced shear strengths exhibited in the end regions of girders with shielded strands. However, -as shown in Figure 43, the test shears for girder AO-25-R, were greater than all the predicted values, and were also greater than the required shear strengths. . According to Table 4-1, the current AASHTO Code provides the best prediction of shear strength but this was still 28% less than the test strength.

For girders with shielded strands, i.e., girders A2-25-3R (Fig. 4-4) and A2-50-3R (Fig. 4-5), in comparison with the companion unshielded specimens, the effect of the shielding reduces the shear capacity predicted by the proposed code with the values for ε_x=0.003, and ε_x computed, approaching the same. Also, the predicted shear capacity values are essentially the same for beam A2-50-3R. In addition, because the shear steel is greatly increased, the shear capacities predicted by- the proposed code (with both ε_x=0.003, and ε_x computed) are greater than the values predicted by the current AASHTO Code. This is clearly shown in both Figures 4-4 and 4-5. However, these latter-figures also show that the test shears are much less than the predicted strengths due to the mode of failure being flexure rather than shear.

It is clear that while the effect of shielding the strands is to reduce the shear capacity, the increase in predicted strength due to the increased shear steel (in girders with 2 and 3 R) is too great. It should be noted that except near midspan, the girders with added shear reinforcement, far exceeds the limits imposed by the current and proposed AASHTO Codes. Such girders tended to fail in flexural mode, with the applied shear at failure section being well above the required shear strength.

4.2 EFFECT OF CONFINEMENT STEEL

The effects of confinement steel can be seen by comparing the results for those girders provided with confinement steel against those not provided with such reinforcement and indicated by the final letter "D" in their designations. For example, the results for girders AO-00-R (with confinement steel) and AO-00-RD (without confinement steel) are shown in Figures 4-1 and 4-6, respectively. As shown in these figures, the predicted shear capacities are not effected by the presence or lack of confinement steel. However, the test results clearly show that test shear strength is reduced when confinement steel is not present.

In Figure 4-1 the values for the tests shears at both ends of AO-00-R are much greater than the predicted capacities, the ratio of the test values to the AASHTO Code values being 1.41 and 1.25 for the TEST NORTH and TEST SOUTH values, respectively. Figure 4-6 shows that the test shears of beam AO-00-RD are greatly reduced in comparison to AO-00-R. The test shears in the former specimen are approximately equal to the current AASHTO predicted values, the ratios of test capacity to current AASHTO capacity being 1.06 and 1.03 for TEST NORTH and TEST SOUTH, respectively (Table 4-1). As shown in Table 4-1, the values of the load position, a, differs somewhat in these tests for AO-00-RD and AO-00-R, the a/L ratios in the corresponding sets are approximately the same.

The results for specimens Al-00-R and A1-00-RD (Figs. 4-7 and 4-8) also show a similar reduction in shear capacity is reduced when confinement steel is not present. The shear capacity for Al-00-R with confinement steel (Figure 4-7) is greater than the capacity predicted by the current AASHTO Code, the test to AASHTO ratios being 1.09 and 1.31 for the TEST NORTH and TEST SOUTH, respectively. However, shear capacity is reduced in beam Al-00-RD, for which,- the ratios of the test capacity to current AASHTO capacity are 0.93 and 1.19, respectively for the TEST NORTH and TEST SOUTH values. For the TEST NORTH values, the presence of confinement steel increases the shear capacity by 17% (from 179K to 210K). Similarly, for the TEST SOUTH values, the presence of confinement steel increases the shear capacity by 10% (from 189K to 208K).

For girders A2-00-3R and A2-00-3RD (Figs. 4-9 and 4-10), the failure mode was that of flexure, since the high amount of shear reinforcement eliminated a shear failure.

4.3 EFFECT OF STRAND SHIELDING

The specimens, AO-25-R, A2-25-3R, A2-50-3R, CO-25-R, CO-50-R, Cl-25-R and C1-50-R contained shielded strands. By comparing Figures 4-1 and 4-3 (AO-00-R and AO-25-R), it is seen that strand shielding reduces the shear capacity predicted by the proposed code (for computed ε_x), but outside of the transfer length, has no effect on the shear capacity predicted by the current AASHTO Code. The test capacity for the north end of AO-25-R is 28% greater than the AASHTO predicted value (TEST NORTH/AASHTO =1.28). This excess strength is less than that for the girder in which all strands are bonded: (AO-00-R), which had a test strength 41% greater than the AASHTO predicted value. However, the reduction in strength due to shielding is not as great as the reduction in strength due to the absence of confinement steel. (1.06).

For specimens A2-25-3R (Fig. 4-4) and A2-50-3R (Fig. 4-5), the high percentage of shear reinforcement provided allowed a flexural failure to develop, before the applied shear force reached the predicted values. However, the values of the applied shear force at failure were in excess of the shear strength required.

In Group C four girders had shielded strands. These girders are CO-25-R, CO-50-R, Cl-25-R and C1-50-R. Shear capacity plots for specimens CO-25-R and CO-50-R are shown in Figures 4-11 and 4-12, respectively. The test shear capacities for these girders are closely predicted by the AASHTO Code. For CO-25-R, the ratios of shear capacities (Test/AASHTO) are 1.11 and 1.18, respectively, for TEST NORTH and TEST SOUTH. Similarly, for CO-50-R, the ratios are 1.09 and 1.22, respectively, for TEST NORTH and TEST SOUTH. The increase of strand shielding from 25% to 50% has the effect of reducing the shear capacity by 2% for the TEST NORTH and increasing the capacity by 3% for the TEST SOUTH. However, it should be pointed out that the mode of failure for the south end of both girders was flexure. Thus, the change in capacity due to increased strand shielding is negligible for girders loaded outside the transfer length.

Girders C1-25-R and Cl-50-R had two load points and a span of 40 ft., thus test values for both ends were for the same load condition. As was the case for beams CO-25-R and CO-50-R, the ratio of test shear values to AASHTO predicted capacities were all greater than 1.0. The values ranged from 1.09 to 1.21, as shown Table 4-1. For TEST NORTH, the increase in strand shielding from 25% to 50% results in a decrease of test shear values of 3.7%. In both girders the failure initiated at the north end which prevented the occurrence of failure at the south end as. can be seen from the crack pattern charts in Appendix A5. Also, it can be seen from the crack charts that the increased shielding resulted in lesser number of cracks an a different failure mode as- indicated in Table 4-1(c).

4.4 EFFECT OF LOW PERCENTAGES OF SHEAR REINFORCEMENT Except for those girders without confinement steel,, most beams provided with less than or equal to 1.5 times the shear steel required by the current AASHTO Code, gave test shear capacities greater than those predicted by that code. The only exception was C1-00-3R/2 which had a north end test shear capacity of 192K which was only 2% less than predicted AASHTO capacity, 195K. This girder developed many flexural cracks. The crack pattern for this girder is shown in Figure 413. For the TEST SOUTH of girder Cl-00-3R/2, ratio of the test capacity to me capacity predicted by the AASHTO code was 1.03.

The south end of Cl-00-R and north end of beam C1-00-3R/2 were subjected to the same load conditions. Comparison- of the shear values for these two beams indicates that the test shear for C1-00-R(S) (196K) is greater than the test shear of beam Cl-00-3R/2(N) (192K). The crack patterns shown in Figs. 4-13 and 4-14 indicate that flexure played a major role in the failure of these beams. However, as shown in Table 4-1, the moments developed during testing exceeded the design flexural capacity of the girders, thereby confirming that the reduction of shear capacity for beam Cl-00-3R/2 is influenced by the flexural mode of failure. This also explains why C1-00-3R/2 was slightly lower than the shear capacity predicted by the AASHTO Code (195K).

4.4.1 Group A1 Girders

Girder Al-00-M was provided with the minimum amount of shear reinforcement permitted by the current AASHTO Code. It is shown in Figure 4-2 that the shear capacities predicted by the proposed code are much lower than the capacities predicted by the current AASHTO Code. The latter code provides the best approximation for the test shear capacities. As shown in the Figure 4-2, the TEST NORTH value was slightly lower than- that required, but was . approximately equal to the prediction of the current AASHTO Code. The ratio of the test value to the AASHTO code value is 1.08 for the north end test. The test shear capacity for the south end was 41% greater than the code prediction. For this same specimen, Al-00-M, the least strength prediction results from applying the proposed code with ε_x=0.003 which predicts a strength of 42K, i.e., one-fourth of the test shear (168K). The plots for girder Al-00-R/2 (Figure 4-15) show that the test strengths are 16% and 34% greater than predicted by the current AASHTO Code, which again gives the best prediction of shear strength. Both TEST NORTH and TEST SOUTH shear strengths are greater than all the predicted capacities, and also greater than the required capacities at the test sections. Furthermore, although, the shear reinforcement provided was only one-half of that required by the current AASHTO Code, girder A1-00-R/2 test results indicate shear strengths which are only slightly lower than the required strengths. This is even more so, than is shown in the Figure 4-15, since the required strength for the current AASHTO code is less than that shown (i.e. V_u/0.9 instead of V_u/0.85).

From the results for girder Al-00-R (Fig. 4-7), it can be seen that the shear capacities predicted by the current code are greater than the required shear capacities, and that both tests shear values are greater than all the predicted shear capacities. The ratios of the test capacity to the AASHTO capacity are, 1.09 and 1.31 for the TEST NORTH and TEST SOUTH, respectively (Table 4-1). For, the south end (TEST SOUTH), the applied load was 124 inches from the centerline of the support, i.e. 0.33L of the span. The shear capacities predicted by all provisions of the proposed code (97K) is less than 50% of the test value of 208K. Thus, the proposed code greatly underestimates the shear strength of this beam which was reinforced for shear in accordance with the current AASHTO code provisions, while the current code gave the best prediction.

Girder A1-00-3R/2 was provided with one and one-half tines the shear reinforcement required by the current AASHTO code. The shear capacity plots for this beam are shown in Figure 4-16. Again, the current AASHTO code provides the best prediction of shear strengths with the TEST NORTH shear capacity being approximately equal to the predicted value. The shear capacity ratios based on the latter code for the NORTH TEST and SOUTH TEST were 1.04 and 1.18, respectively.

; versus predicted shear strength based on the current AASHTO .es girders are shown plotted in Figure 4-17 including those onfinement steel. It is seen that, except for one test of the D hich failed in flexure), all the test shear strengths exceeded ASHTO provisions. On the other hand, the provisions of the to underestimate shear strength (see Table 4-2) and can be e.

Girders

girders C0-00-RD(N) and C1-00-3R/2(N) had shear strengths nt AASHTO Code. The ratios of test to predicted values are rs, respectively. Again, the beneficial effect of confinement beams (RD) not provided with such reinforcement tended to the companion specimens which contained confinement steel. 00-3R/2 shown in Fig. 4-13 and the ratios of M/M_n for this at flexure played a major role in the failure of this beam, low strength ratios of this girder in comparison to the others

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h were provided with shear reinforcement two or three times xcept for test A2-00-2R(S) the shear applied to these girders predicted by both the current and the proposed AASHTO he test shear capacity was approximately equal to the current ; The shear ratios ranged from 1.06 for current AASHTO to 4-1). However, in this test , the a/d ratio was lower than l, the shear force was closer to the strength limit of the -18).

The test shear values of girder A2-00-3R, shown in Fig. 4-9 fall between the limits recommended by the two codes (AASHTO and the proposed code). These shear values are much less than the predicted shear strengths, which is due to the fact that the beam failed in flexure rather than shear.

4.5.2 Group B0 Girders

Groups B0 and B1 girders include specimens which were provided with two and three times the amount of shear reinforcement required by design. The plots for shear capacities for girders BO-00-R, BO-00-2R and BO-00-3R are shown in Figures 4-19 to 4-21. Figure 4-19 and Table 4-1(b) show that the test results for Beam BO-00-R are greater than the predicted shear values and that the current AASHTO code provides the best predictions of shear strengths.

Figure 4-20 shows that the test shear values for Beam BO-00-2R which failed in flexure are lower than the values predicted by the current AASHTO Code, the ratio of test values to AASHTO values being 0.84 and 0.97 for TEST NORTH and TEST SOUTH, respectively. For, the proposed AASHTO Code, the ratio of the test shear capacity to the proposed code value is 1.19 for TEST SOUTH. By Comparing the test shears for girders BO-00-R and BO-00-2R it is seen that the relative increase in test shears due to doubling the amount of shear steel is small, and functions mainly to transform from a shear failure mode to a flexural failure. The same is also true for girder BO-00-3R. This indicates that for shear reinforcement greatly in excess of the amount required by AASHTO, although not warranted from the point of view of shear, may be useful in guaranteeing the more desirable flexural mode of failure. The moments developed in these beams ranged from 89% to 114% of the design flexure capacity (Table 4-1(b)).

A comparison of the crack patterns for girders BO-00-R, BO-00-2R and BO-00-3R (Figures 4-22 to 4-24) indicates that as additional shear reinforcement is provided, the shear strength of the girder and the number of flexure cracks, as an indication of the enhanced ductility, are increased.

4.5.3 Group B1 Girders

Group B 1 girders also demonstrate the effect of increased shear reinforcement. The girders in this group were 21 ft. versus 41 ft. for all girders in other groups. This group was designed to investigate only the shear behavior of the girders. Therefore all tests were conducted with very short shear span as shown in Table 4-1. Also, the shear span/depth ratio was much lower than group BO girders due to the shorter span length. All the B1 girders failed in shear, with typical diagonal shear cracks developing with simultaneous strand slip. The additional load applied to the girders after the initial shear cracks varied depending on the amount of provided shear reinforcement. The crack patterns at failure for these girders are shown in Figures 4-25 to 4-28.

Girder Bl-00-0R contained no shear reinforcement. The predicted shear capacities for this beam are shown in Figure 4-29. This figure along with Table 4-1(b) indicate that the codes greatly under-predict the shear contribution of the concrete, V_c, to the overall shear strength. The current AASHTO code gives the best approximation, but even this value is an average of only 54% of the test value. The least prediction of V_c is provided by the proposed AASHTO Code provisions with an assumed longitudinal strain , $\varepsilon_X = 0.003$, which predicts a very low shear strength of 14K, i.e., less than 10% of the test value.

The shear capacity plots for girders BI-00-R, B l-00-2P, B l-00-2R(2), and BI-00-3R are shown in Figures 4-30 to 4-33. As shown in Figure 4-30, for Bl-00-R, the current AASHTO code gives the closest prediction of test results; the shear capacity ratios being 1.16 and 1.10 for the TEST NORTH and TEST SOUTH, respectively.

Figures 4-30 and 4-31 show that the test shears for two identical girders BI-00-2R and B1-00-2R(2) which were subjected to the same loading were approximately the same. For the test values versus the current AASHTO values, the average shear capacity ratios were 0.79 and 0.76 (ignoring the code specified limit for shear reinforcement) for the north and south ends, respectively. For the two beams, the average test shear capacities were 265K and 251K for TEST NORTH and TEST SOUTH, respectively.

Doubling of the shear reinforcement between girders B 1-00-R and BI-00-2R resulted, in an increase in shear strength of only 8%, at both ends. For Bl-00-2R and B1-00-2R(2), the test shear capacities were slightly greater than the shear capacity limits proposed by the current AASHTO Code. The average ratios of the test shear strength to predicted shear strength were 0.78, 0.51 and 0.74 for the CURRENT AASHTO, PROPOSED AASHTO, and PROPOSED AASHTO with ε_x=0.003, respectively. These ratios were computed using predicted shear capacities (averaged for both TEST NORTH and TEST SOUTH) ignoring the upper code limits. If the code limits are imposed, the shear capacities would be limited to 239 Kips for the current AASHTO code and 351 Kips for the proposed code, and would result in the shear capacity ratios being 1.08, 0.74 and 0.74 for CURRENT AASHTO, PROPOSED AASHTO, and PROPOSED AASHTO with ex =0.003, respectively. This justifies the imposition of limits on the shear strength due to. shear reinforcement and, in this case, the current AASHTO limit best approximates the test results.

The crack patterns for these girders (Figures 4-26 to 4-28) indicate that shear cracking was concentrated in a very narrow band. Shear failure occurred soon after shear cracking commenced, and increased shear reinforcement had very little effect on the failure shear values. Figures 4-30, 4-31 and 4-33 show that if the imposed limits are ignored, the shear capacities proposed by the codes increase as the amount of shear reinforcement increases. For beam B l-00-3R (Fig. 4-33), the current AASHTO Code predicts the lowest shear capacity values. However, the test shear values are much less than the predicted values since the excessive amount of shear reinforcement was above the code limit. The increased shear reinforcement resulted in enhanced ductility. Again, it is seen that for shear reinforcement greater than the amount required by AASHTO, large increases in shear reinforcement are not necessarily accompanied by large increases in shear strength.

For girder B1-00-0R, the concrete shear contribution, V_c , at the north and south ends were 166K and 155K , respectively. Since the shear span was constant for all girders in this group, the concrete shear strength, V_c, for girder B1-00-OR can be used in calculating the actual shear reinforcement contribution, V_s for beams B1-00-R, BI-00-2R, B l-00-2R(2), and B l-00-3R. Utilizing the value of V_c for the north end test (165K) the values of V_s can be obtained be subtracting V_c from the actual shear at failure. The resulting V_s values are 79K, 96K, 102K and 98K for girders B l-00-R, B l-00-2R, B l-00-2R(2), and B l-00-3R, respectively. Similarly, for the south end test with V_c =155K , the V_s values are 77K, 92K, 100K, and 108k, respectively. By averaging the values for the north end and the south end and also averaging the values for girders B l-00-2R and B l-00-2R(2), the average V_s values based on the current AASHTO Code are as follows:

Girder B1-00-R, Vs = 78K

Girder B1-00-2R, Vs =98K

Girder B1-00-3R, Vs =103K.

Thus, increasing the shear reinforcement by 100% (i.e., from R to 2R) results in only a 25% increase in V_s. Further increase from 200% to 300% (i.e., 2R .to 3R) results in only 5% enhancement in shear strength provided by the steel reinforcement.

4.6 USE OF WIRE MESH AS SHEAR REINFORCEMENT

In girders A3-00-RA and A3-00-RB (Table 4-1(a)) a welded wire mesh was fabricated and used as shear reinforcement. The span and load points for girders A3-00-RA and A3-00-RB were the same used in Al-00-R and AO-00-R, respectively. The vertical bars of the wire mesh provided shear reinforcement approximately equivalent to that provided in girders Al-00-R and AO-00-R, therefore, except for A3-00-RA the predicted shear capacities were approximately the same at the load points. The predicted shear capacities for A3-00-RA are shown in Fig. 4-34. It can be seen that the current AASHTO Code underestimates the shear capacity but still provides a closer prediction of shear strength than the proposed AASHTO Code.

For girders A3-00-RA and Al-00-R, the test loads were applied , respectively, at 102 in. (0.21 L) and 124 in. (0.33 L) from the centerline of the support for the TEST NORTH and TEST SOUTH. From a comparison of these tests, there is a noticeable increase in the shear capacity of girder A3-00-RA. This girder was subjected to shears equal to 271K and 232K. for TEST NORTH and TEST SOUTH, respectively, which were 60% and 41% greater than the current AASHTO predicted values. In addition, the moments developed in this girder were 13% and 18% greater than the required design flexural capacity. The test shears in Al-00-R were 210K and 208K which were 9% and 31% greater than the shear capacities predicted by the current AASHTO Code. Therefore, girder A3-00-RA had shear capacities 29% and 12% greater than those of Al-00-R. Table 4-1 shows that the shear capacities predicted for the two girders were different at the load point, due to a difference in shear reinforcement details at this location. The crack patterns for girder A3-00-RA are shown in Figure 4-35.

Girders A3-00-RB and AO-00-R were both loaded at 85 in. and 124 in. (0.18L and 0.26L) from the centerline of the end supports. In the north end test, A3-00-RB failed in flexure. However, the shear value (297K) reached during the test exceeded the AASHTO predicted value (221K) by 34%. In TEST SOUTH, shear played a greater role in the failure mode, the shear capacity being 275K, which was 24% greater than the current AASHTO predicted value. The shear capacity plots for beam A3-00-RB are shown in Fig. 4-36, which shows that the current AASHTO Code predicts a much lower shear capacities than those in the tests. The crack patterns for beam A3-00-RB are shown in Figure 4-37.

A comparison of the results for girders A3-00-RB and AO-00-R, shows that the shear forces developed in A3-00-RB for TEST NORTH (297K) were 5% less than those developed in AO-00-R (313K). This is due to the fact that flexure was the mode of failure in A3-00-RB. For TEST SOUTH, the shear capacity of the two girders were essentially the same.

Table 4-1 (a) : Results of Test Program (A Series Girders)

BEAM	END	L (in)	a (in)	a/d	a/L	TEST V (k)	V SLIP (k)	PREDICTED SHEAR CAPACITY (kips)								M/Mn	FAILURE	@ldpt TEST M (k-ft)
								CURRENT AASHTO		PROPOSED AASHTO		PROPOSED ϵ_s=0.003		RESPONSE PROGRAM				
									(7)/(9)		(7)/(11)		(7)/(13)		(7)/(15)			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
A0-00-R	N	480.	85.	2.1	0.18	313.	259	221.	1.41	196.	1.60	180.	1.74	256.	1.22	1.09	SHEAR/BOND	2237.
A0-00-R	S	324.	85.	2.1	0.26	276	NS	221.	1.25	196.	1.41	180.	1.53	256.	1.08	0.96	SHEAR	1975.
A0-00-RD	N	480.	74.	1.8	0.15	230.	207	217.	1.06	204.	1.13	180.	1.28	256.	0.90	0.69	SHEAR/BOND	1433.
A0-00-RD	S	306.	85.	2.1	0.28	228.	NA	221.	1.03	196.	1.17	180.	1.27	256.	0.87	0.79	SHEAR/BOND	1634.
A1-00-M	N	480.	102.	2.5	0.21	141.	129	130.	1.08	57.	2.47	51.	2.76	112.	1.26	0.59	SHEAR/BOND	1223.
A1-00-M	S	378.	124.	3.1	0.33	168.	135	119.	1.41	42.	4.00	42.	4.00	112.	1.50	0.86	SHEAR/BOND	1780.
A1-00-R/2	N	480.	102.	2.5	0.21	166.	136	143.	1.16	76.	2.18	69.	2.41	129.	1.29	0.70	SHEAR/BOND	1437.
A1-00-R/2	S	378.	124.	3.1	0.33	173.	162	129.	1.34	55.	3.14	55.	3.14	112.	1.54	0.89	SHEAR/BOND	1830.
A1-00-R	N	480.	102.	2.5	0.21	210.	144	193.	1.09	148.	1.42	138.	1.52	219.	0.96	0.88	SHEAR/BOND	1809.
A1-00-R	S	378.	124.	3.1	0.33	208.	190	159.	1.31	97.	2.14	97.	2.14	168.	1.24	1.06	FLEXURE/BOND	2191.
A1-00-3R/2	N	480.	102.	2.5	0.21	207.	161	200.	1.04	157.	1.31	147.	1.41	227.	0.91	0.87	SHEAR/BOND	1785.
A1-00-3R/2	S	378.	124.	3.1	0.33	230.	185	195.	1.18	147.	1.56	147.	1.56	227.	1.01	1.17	FLEXURE	2413.
A1-00-RD	N	480.	102.	2.5	0.21	179.	152	193.	0.93	148.	1.22	138.	1.30	219.	0.83	0.75	SHEAR/BOND	1551.
A1-00-RD	S	378.	124.	3.1	0.33	189.	151	159.	1.19	97.	1.95	97.	1.95	168.	1.13	0.97	SHEAR/BOND	1997.
A2-00-2R	N	480.	102.	2.5	0.21	257.	241	264.	0.97	249.	1.03	235.	1.09	297.	0.87	1.07	FLEXURE	2212.
A2-00-2R	S	378.	74.	1.8	0.20	357.	282	338.	1.06	376.	0.95	346.	1.03	364.	0.98	1.07	FLEXURE	2218.
A2-00-3R	N	480.	102.	2.5	0.21	257.	NS	352.	0.73	375.	0.69	357.	0.72	370.	0.69	1.07	FLEXURE	2210.
A2-00-3R	S	378.	85.	2.1	0.22	312.	273	474.	0.65	558.	0.55	529.	0.58	452.	0.68	1.08	FLEXURE	2228.
A2-00-3RD	N	480.	102.	2.5	0.21	254.	247	352.	0.72	375.	0.68	357.	0.71	370.	0.69	1.06	FLEXURE	2187.
A2-00-3RD	S	378.	74.	1.8	0.20	333.	294	471.	0.71	565.	0.59	529.	0.63	452.	0.74	1.00	FLEXURE/BOND/SHEAR	2066.

NA = NOT AVAILABLE NS = NO SLIP

Table 4-1 (a) : Results of Test Program (A Series Girders) (continued)

BEAM	END	L (in)	a (in)	a/d	a/L	TEST V (k)	V SLIP (k)	PREDICTED SHEAR CAPACITY (kips)								M/Mn	FAILURE	@ldpt TEST M (k-ft)
								CURRENT AASHTO		PROPOSED AASHTO		PROPOSED ϵ_x=0.003		RESPONSE PROGRAM				
									(7)/(9)		(7)/(11)		(7)/(13)		(7)/(15)			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
A0-25-R	N	480.	79.	2.0	0.16	281.	180	219.	1.28	196.	1.43	180.	1.56	256.	1.10	1.06	SHEAR/BOND	1863.
A0-25-R	S	378.	150.	3.7	0.40	173.	147	139.	1.24	78.	2.22	78.	2.22	145.	1.19	1.08	BOND/FLEXURE	2220.
A2-25-3R	N	480.	79.	2.0	0.16	253.	191	472.	0.54	558.	0.45	529.	0.48	452.	0.56	0.96	FLEXURE/BOND	1684.
A2-25-3R	S	378.	150.	3.7	0.40	170.	NS	241.	0.71	203.	0.89	203.	0.84	286.	0.59	1.05	FLEXURE	2177.
A2-50-3R	N	480.	79.	2.0	0.16	200.	180	472.	0.42	553.	0.36	529.	0.38	452.	0.44	0.91	FLEXURE/BOND	1332.
A2-50-3R	S	378.	150.	3.7	0.40	160.	149	241.	0.66	203.	0.79	203.	0.79	286.	0.56	1.00	FLEXURE	2060.
A3-00-RA	N	480.	102.	2.5	0.21	271.	203	169.	1.60	114.	2.38	105.	2.58	180.	1.51	1.13	FLEXURE/BOND	2334.
A3-00-RA	S	378.	124.	3.1	0.33	232.	204	165.	1.41	105.	2.21	105.	2.21	180.	1.29	1.18	FLEXURE/BOND	2441.
A3-00-RB	N	480.	85.	2.1	0.18	297.	223	221.	1.34	196.	1.52	180.	1.65	256.	1.16	1.03	FLEXURE/BOND	2120.
A3-00-RB	S	324.	85.	2.1	0.26	275.	195	221.	1.24	196.	1.40	180.	1.53	256.	1.07	0.95	SHEAR/BOND/FLEXURE	1969.

NS = NO SLIP

Table 4-1 (b) : Results of Test Program (B Series Girders)

BEAM	END	L (in)	a (in)	a/d	a/L	TEST V (k)	V SLIP (k)	PREDICTED SHEAR CAPACITY (kips)								M/Mn	FAILURE	@ldpt TEST M (k-ft)
								CURRENT AASHTO		PROPOSED AASHTO		PROPOSED ϵ_x=0.003		RESPONSE PROGRAM				
									(7)/(9)		(7)/(11)		(7)/(13)		(7)/(15)			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
B0-00-R	N	480.	102.	2.5	0.21	220.	185	194.	1.13	149.	1.48	140.	1.57	221.	1.00	0.89	SHEAR/BOND/FLEXURE	1899.
	S	378.	124.	3.1	0.33	206.	NS	161.	1.28	98.	2.10	98.	2.10	169.	1.22	1.02	SHEAR /FLEXURE	2170.
B0-00-2R	N	480.	102.	2.5	0.21	223.	182	265.	0.84	250.	0.89	237.	0.94	300.	0.74	0.90	FLEXURE/BOND	1919.
	S	378.	124.	3.1	0.33	216.	195	222.	0.97	181.	1.19	181.	1.19	260.	0.83	1.07	FLEXURE	2274.
B0-00-3R	N	480.	102.	2.5	0.21	231.	194	354.	0.65	377.	0.61	359.	0.64	374.	0.62	0.94	FLEXURE/BOND	1993.
	S	378.	124.	3.1	0.33	236.	216	289	0.82	273.	0.86	273.	0.86	324.	0.73	1.14	FLEXURE	2420.
B1-00-2R	N	240.	60.	1.5	0.25	262.	243	334.	0.78	481.	0.55	348.	0.75	368.	0.71	0.70	SHEAR/BOND	1318.
	S	222.	54.	1.3	0.24	247.	232	331.	0.75	548.	0.45	348.	0.71	368.	0.67	0.63	SHEAR/BOND	1120.
B1-00-3R	N	240.	60.	1.5	0.25	264.	233	467.	0.57	723.	0.37	532.	0.50	457.	0.58	0.71	SHEAR/BOND	1329.
	S	222.	54.	1.3	0.24	263.	243	465.	0.57	824.	0.32	532.	0.49	457.	0.58	0.66	SHEAR/BOND	1191.
B1-00-0R	N	240.	60.	1.5	0.25	166.	133	90.	1.84	43.	3.86	11.	15.09	66.	2.52	0.45	SHEAR/BOND	838.
	S	222.	54.	1.3	0.24	155.	147	88.	1.76	71.	2.18	11.	14.09	66.	2.35	0.39	SHEAR/BOND	704.
B1-00-R	N	240.	60.	1.5	0.25	245.	228	212.	1.16	261.	0.94	181.	1.35	260.	0.94	0.66	SHEAR/BOND	1235.
	S	222.	54.	1.3	0.24	232.	215	210.	1.10	297.	0.78	181.	1.28	260.	0.89	0.59	SHEAR/BOND	1052.
B1-00-2R2	N	240.	60.	1.5	0.25	268.	238	334.	0.80	481.	0.56	348.	0.77	368.	0.73	0.72	SHEAR/BOND	1348.
	S	222.	54.	1.3	0.24	255.	237	331.	0.77	548.	0.47	348.	0.73	368.	0.69	0.65	SHEAR/BOND	1157.

NS = NO SLIP

Table 4-1 (c) : Results of Test Program (C Series Girders)

BEAM	END	L (in)	a (in)	a/d	a/L	TEST V (k)	V SLIP (k)	PREDICTED SHEAR CAPACITY (kips) CURRENT AASHTO		PROPOSED AASHTO		PROPOSED $\epsilon_s=0.003$		RESPONSE PROGRAM		M/Mn	FAILURE	@ldpt TEST M (k-ft)
									(7)/(9)		(7)/(11)		(7)/(13)		(7)/(15)			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
C0-00-R	N	336.	142.	3.5	0.42	176.	NS	144.	1.22	79.	2.23	79.	2.23	146.	1.21	1.05	FLEXURE	2139.
	S	480.	132.	3.2	0.28	180.	NS	158.	1.14	94.	1.91	94.	1.91	170.	1.06	0.99	FLEXURE	2021.
C0-00-RD	N	264.	60.	1.5	0.23	189.	161	213.	0.89	235.	0.80	182.	1.04	256.	0.74	0.57	SHEAR/BOND	957.
	S	480.	148.	3.6	0.31	158.	NS	148.	1.07	79.	2.00	79.	2.00	146.	1.08	0.99	FLEXURE /SHEAR	2011.
C0-25-R	N	264.	60.	1.5	0.23	237.	178	213.	1.11	198.	1.20	182.	1.30	256.	0.93	0.88	SHEAR/BOND	1192.
	S	480.	194.	4.8	0.40	123.	NS	104	1.18	65.	1.89	65.	1.89	129.	0.95	1.02	FLEXURE	2080.
C0-50-R	N	264.	60.	1.5	0.23	233.	158	213.	1.09	198.	1.18	182.	1.28	256.	0.91	1.03	SHEAR/BOND	1175.
	S	480.	194.	4.8	0.40	127.	112	104.	1.22	65.	1.95	65.	1.95	129.	0.98	1.05	FLEXURE/BOND	2143.
C1-00-R	N	480.	142.	3.5	0.30	177.	NS	144.	1.23	79.	2.24	79.	2.24	146.	1.21	1.06	FLEXURE	2148.
	S	480.	132.	3.2	0.28	196.	NS	158.	1.24	94.	2.09	94.	2.09	170.	1.15	1.08	FLEXURE	2197.
C1-00-RD	N	480.	149.	3.7	0.31	171.	NS	141.	1.21	79.	2.16	79.	2.16	146.	1.17	1.07	FLEXURE	2185.
	S	378.	149.	3.7	0.39	152.	NS	141.	1.08	79.	1.92	79.	1.92	146.	1.04	0.96	FLEXURE	1949.
C1-00-3R/2	N	480.	132.	3.2	0.28	192.	NS	195.	0.98	139.	1.38	139.	1.38	228.	0.84	1.06	FLEXURE	2159.
	S	480.	126.	3.1	0.26	202.	NS	196.	1.03	149.	1.36	149.	1.36	228.	0.89	1.07	FLEXURE	2166.
C1-25-R*	N	480.	140.	3.4	0.29	164.	146	145.	1.13	79.	2.08	79.	2.08	146.	1.12	0.97	SHEAR/BOND/FLEXURE	1966.
	S	480.	172.	4.2	0.36	143.	NS	118.	1.21	69.	2.07	69.	2.07	129.	1.11	1.05	FLEXURE	2129.
C1-50-R*	N	480.	140.	3.4	0.29	158.	138	145.	1.09	79.	2.00	79.	2.00	146.	1.08	0.93	SHEAR/BOND	1893.
	S	480.	172.	4.2	0.36	133.	115	118.	1.13	69.	1.93	69.	1.93	129,	1.03	0.97	FLEXURE./BOND	1975.

* TWO POINT LOADING

NS = NO SLIP

TABLE 4-2. *COMPARISON OF SHEAR STRENGTHS $\frac{V_{TEST}}{V_n}$

GIRDER SERIES	STATISTICS	CURRENT AASHTO	PROPOSED AASHTO $\epsilon_x = 0.003$	PROPOSED AASHTO
A	MEAN	1.17	1.99	1.88
	STANDARD DEVIATION	0.15	0.85	0.87
	VARIANCE	0.024	0.72	0.76
	STANDARD ERROR	0.043	0.24	0.24
	MAXIMUM	1.41	4.00	4.00
	MINIMUM	0.93	1.27	1.13
B	MEAN	1.01	3.58	1.05
	STANDARD DEVIATION	0.46	5.82	1.13
	VARIANCE	0.21	33.84	1.27
	STANDARD ERROR	0.15	1.84	0.36
	MAXIMUM	1.84	15.09	3.86
	MINIMUM	0.57	0.49	0.32
C	MEAN	1.06	1.41	1.32
	STANDARD DEVIATION	0.11	0.41	0.54
	VARIANCE	0.012	0.17	0.29
	STANDARD ERROR	0.056	0.21	0.27
	MAXIMUM	1.13	2.00	2.08
	MINIMUM	0.89	1.04	0.80
ALL	MEAN	1.10	2.49	1.49
	STANDARD DEVIATION	0.30	3.58	0.99
	VARIANCE	0.091	12.82	0.98
	STANDARD ERROR	0.058	0.69	0.19
	MAXIMUM	1.84	15.09	4.00
	MINIMUM	0.57	0.49	0.32

*FOR BEAMS FAILING IN SHEAR

SHEAR CAPACITY COMPARISON
A0-00-R
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

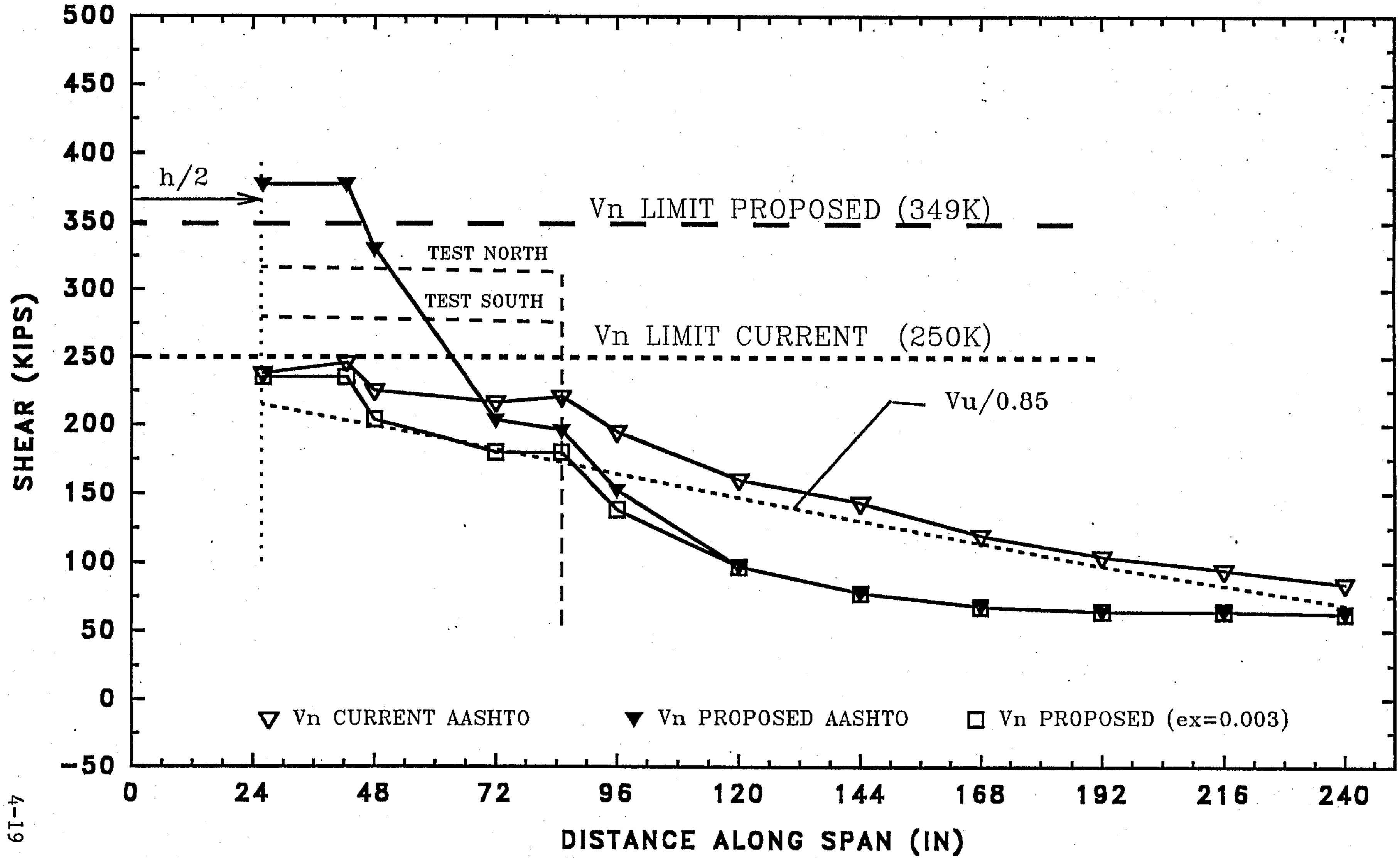


FIGURE 4-1

SHEAR CAPACITY COMPARISON
A1-00-M
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

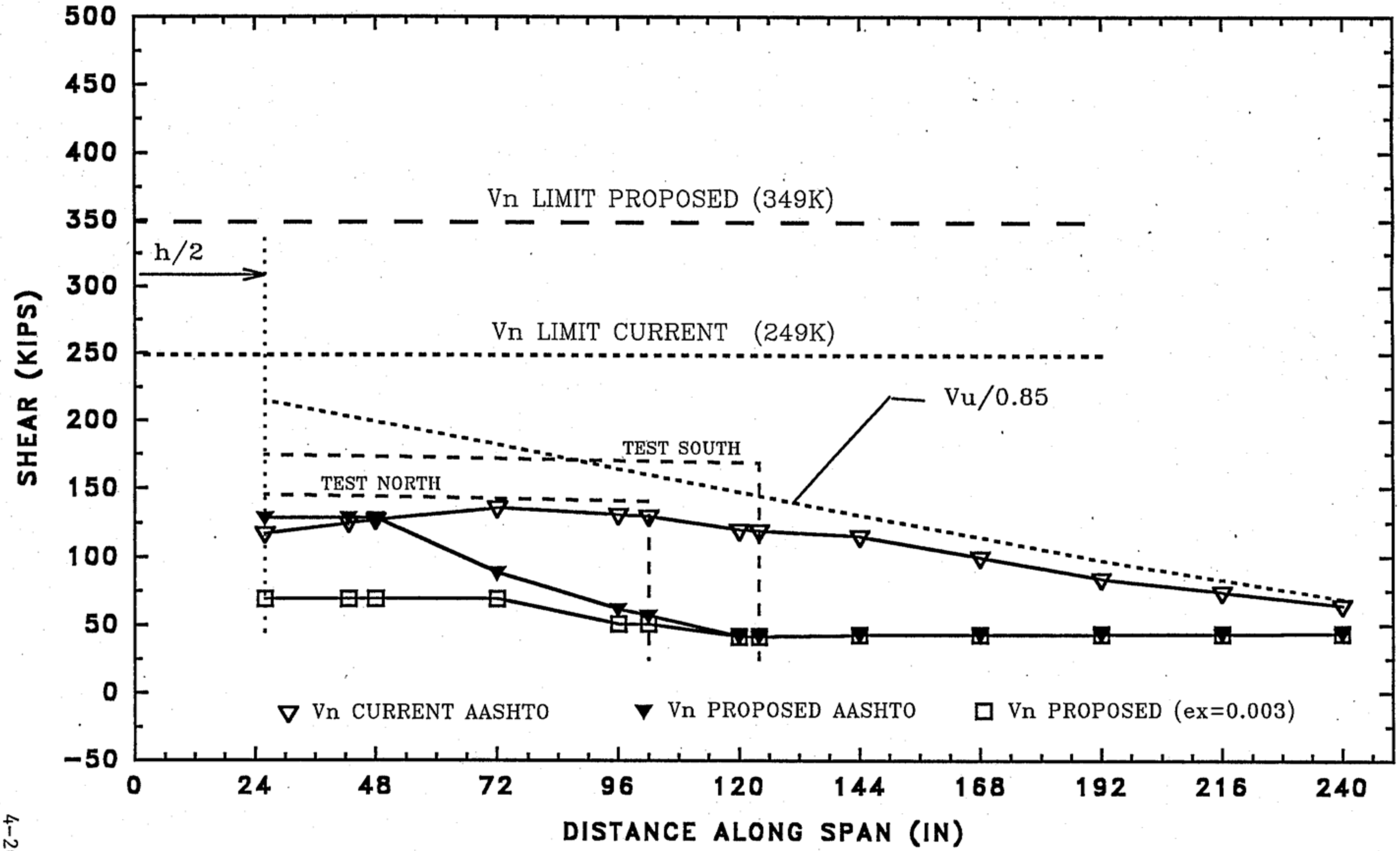


FIGURE 4-2

SHEAR CAPACITY COMPARISON
A0-25-R
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

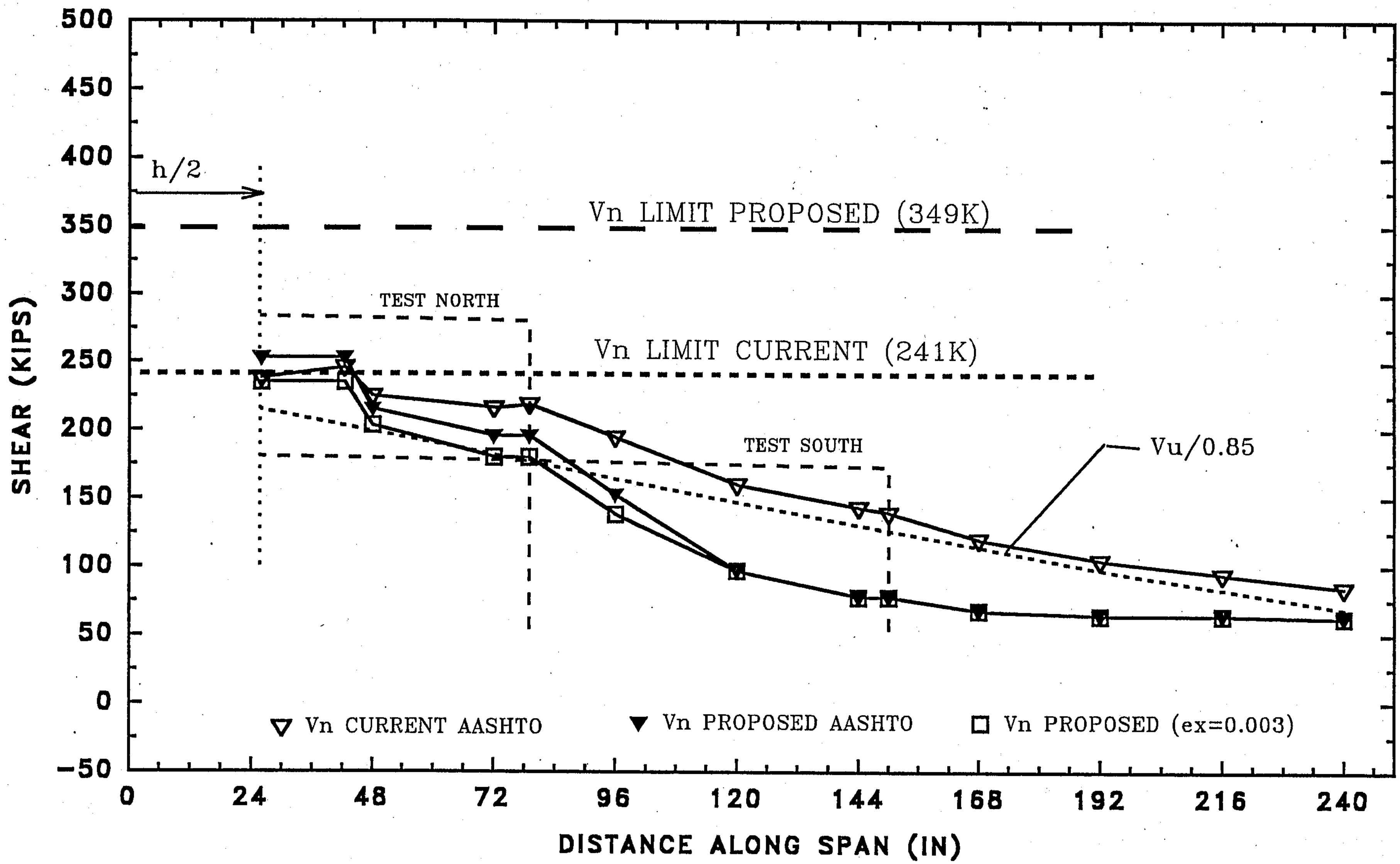


FIGURE 4-3

SHEAR CAPACITY COMPARISON
A2-25-3R
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

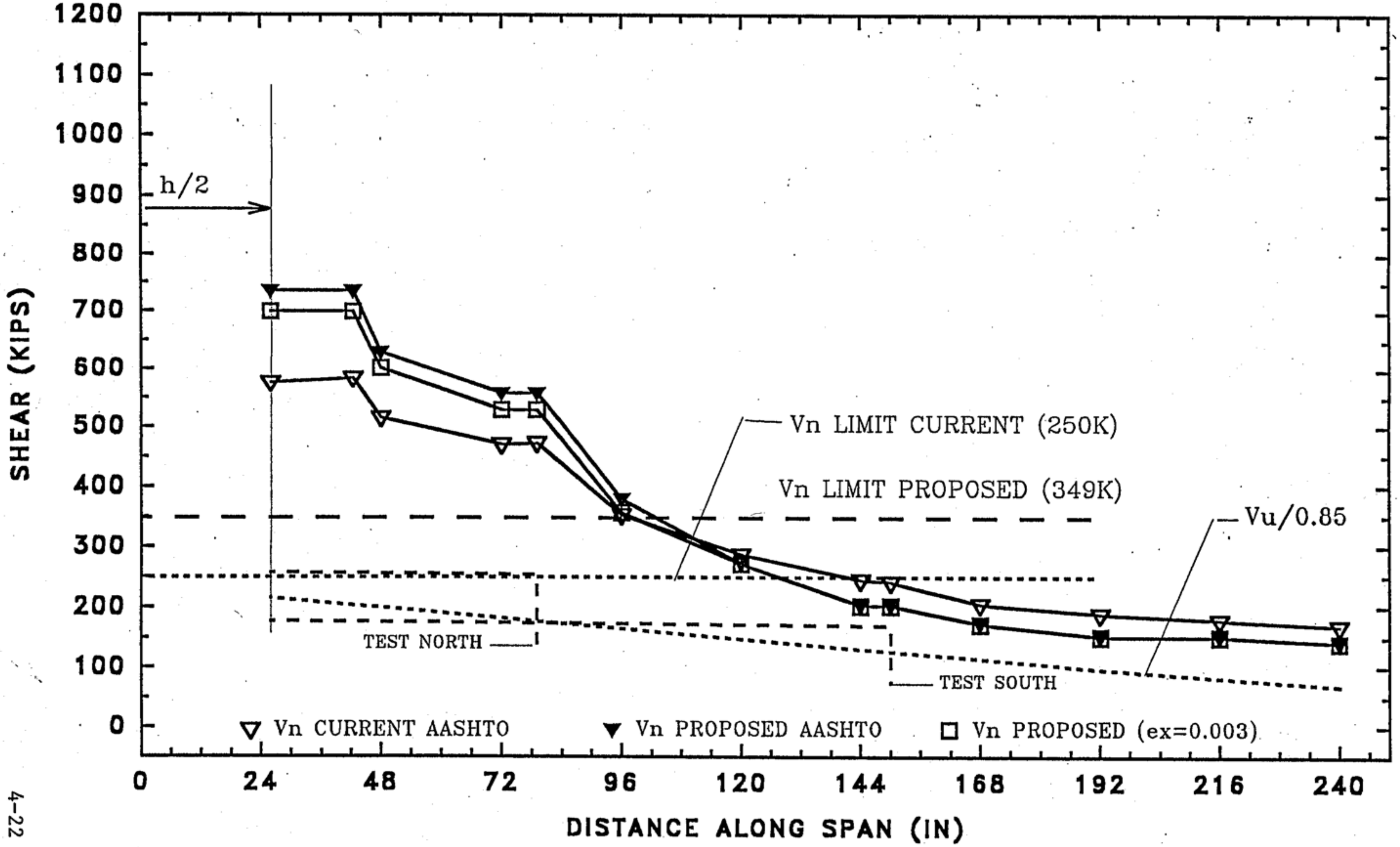


FIGURE 4-4

SHEAR CAPACITY COMPARISON
A2-50-3R
TYPE II BEAM   DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT.   SLAB = 8.00 IN.
F'C SLAB = 6000 PSI   F'C BEAM = 6000. PSI

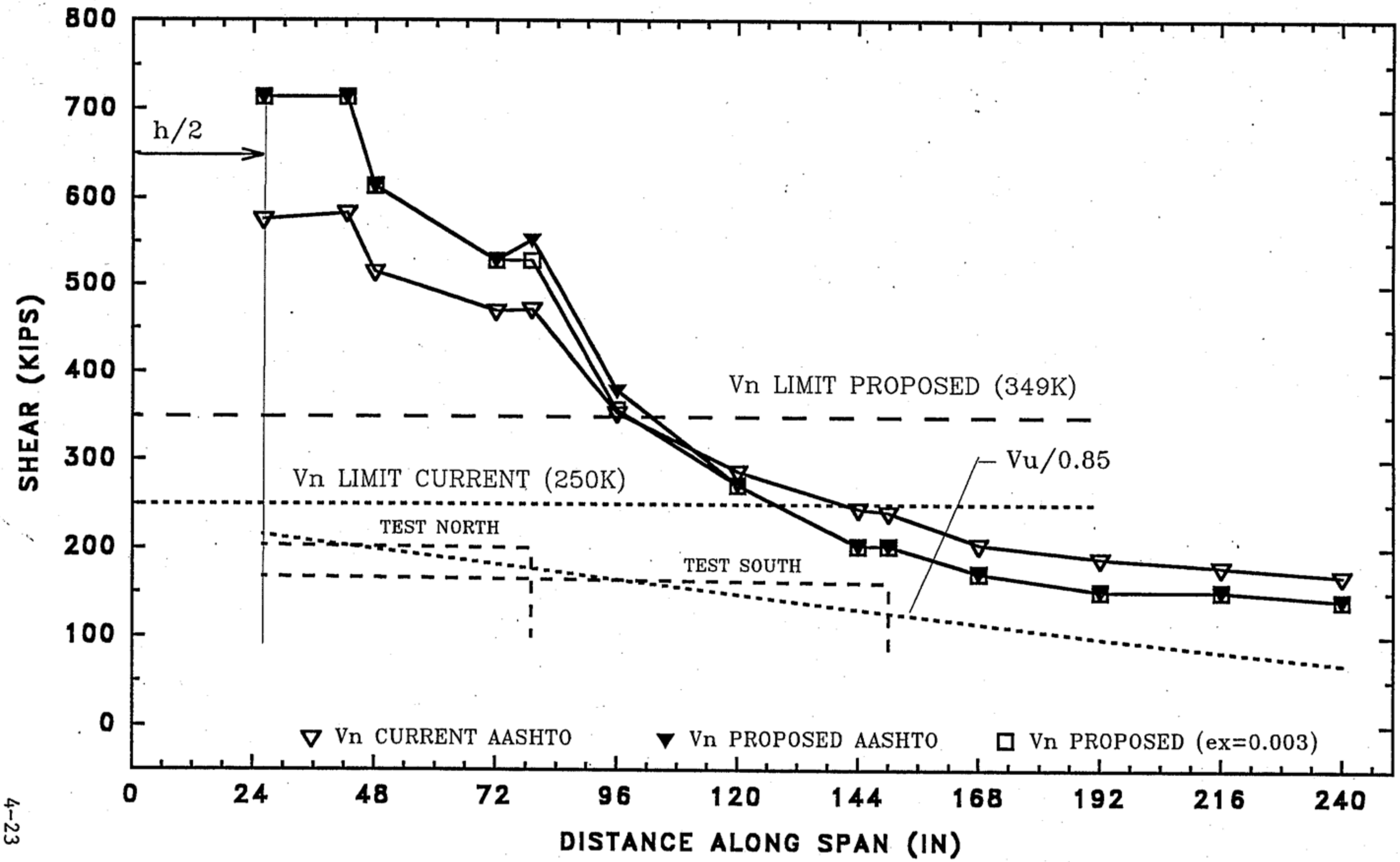


FIGURE 4-5

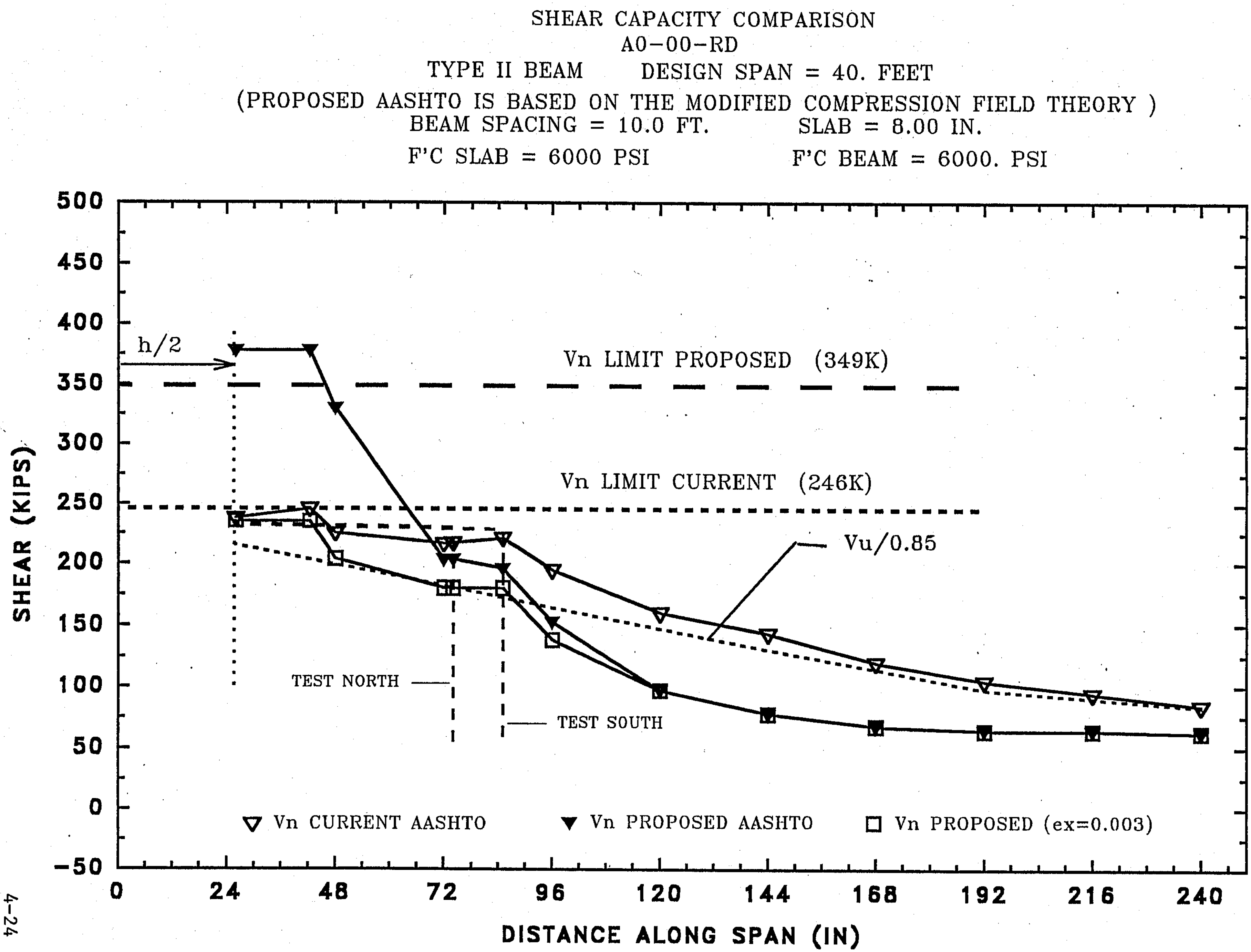

SHEAR CAPACITY COMPARISON
A0-00-RD
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI
SHEAR (KIPS)
500
450
400
350
300
250
200
150
100
50
0
-50
h/2
Vn LIMIT PROPOSED (349K)
Vn LIMIT CURRENT (246K)
Vu/0.85
TEST NORTH
TEST SOUTH
Vn CURRENT AASHTO
Vn PROPOSED AASHTO
Vn PROPOSED (ex=0.003)
0
24
48
72
96
120
144
168
192
216
240
DISTANCE ALONG SPAN (IN)

FIGURE 4-6

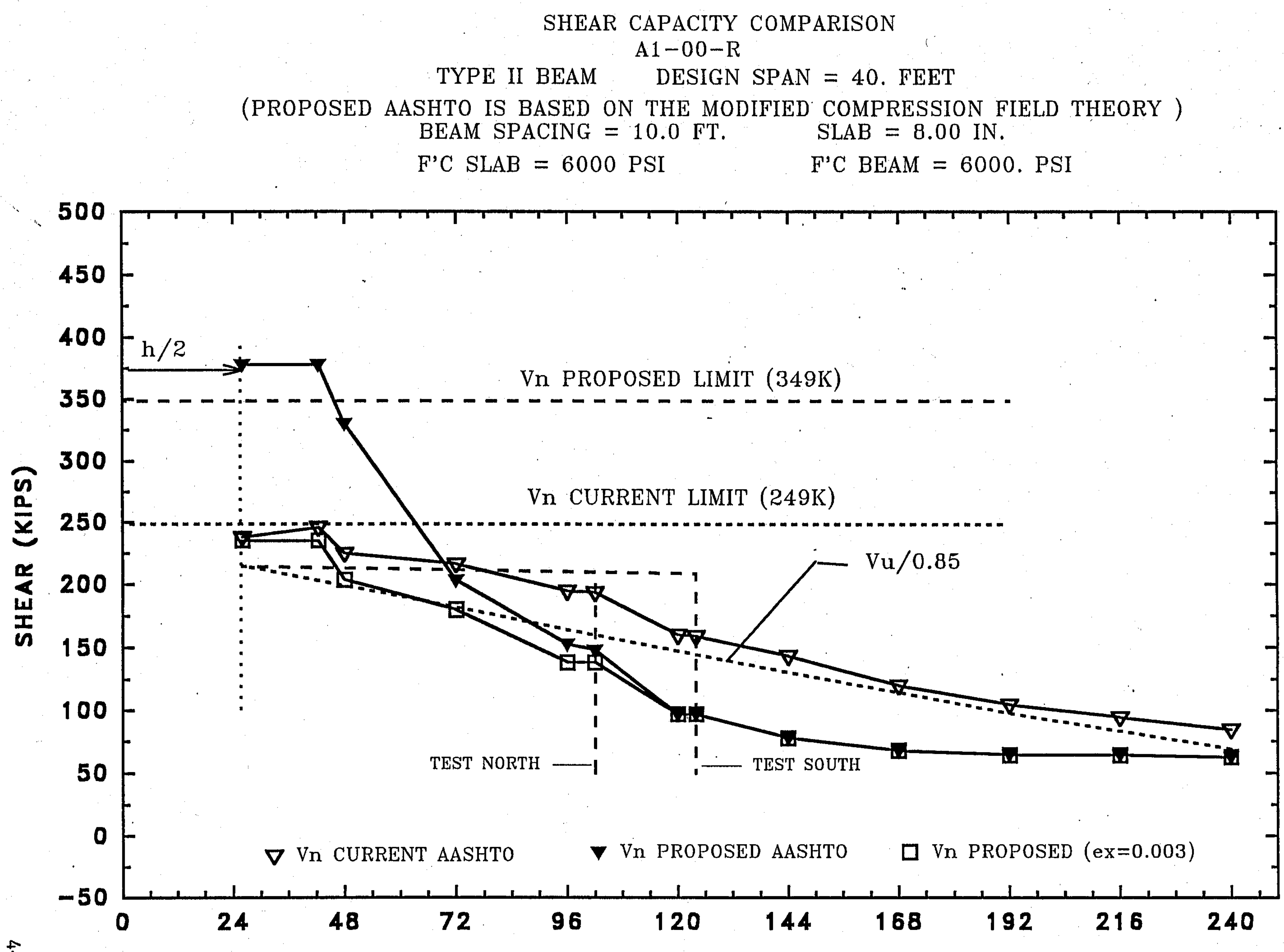
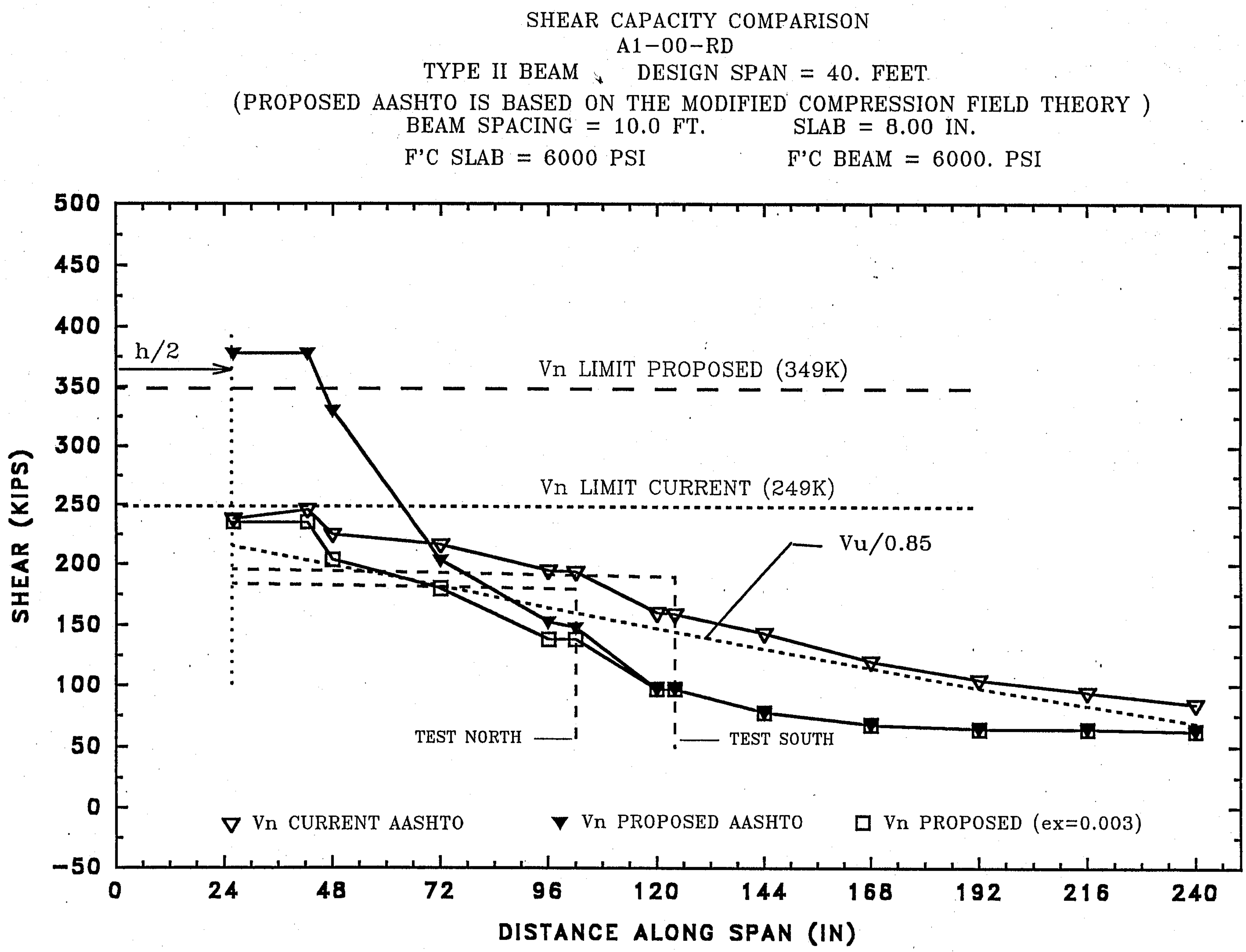

SHEAR CAPACITY COMPARISON
A1-00-R
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI
h/2
Vn PROPOSED LIMIT (349K)
Vn CURRENT LIMIT (249K)
Vu/0.85
TEST NORTH
TEST SOUTH
Vn CURRENT AASHTO
Vn PROPOSED AASHTO
Vn PROPOSED (ex=0.003)
SHEAR (KIPS)
DISTANCE ALONG SPAN (IN)
500
450
400
350
300
250
200
150
100
50
0
-50
0
24
48
72
96
120
144
168
192
216
240

FIGURE 4-7



SHEAR CAPACITY COMPARISON
A1-00-RD
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI
SHEAR (KIPS)
500
450
400
350
300
250
200
150
100
50
0
-50
h/2
Vn LIMIT PROPOSED (349K)
Vn LIMIT CURRENT (249K)
Vu/0.85
TEST NORTH
TEST SOUTH
Vn CURRENT AASHTO
Vn PROPOSED AASHTO
Vn PROPOSED (ex=0.003)
0
24
48
72
96
120
144
168
192
216
240
DISTANCE ALONG SPAN (IN)

FIGURE 4-8

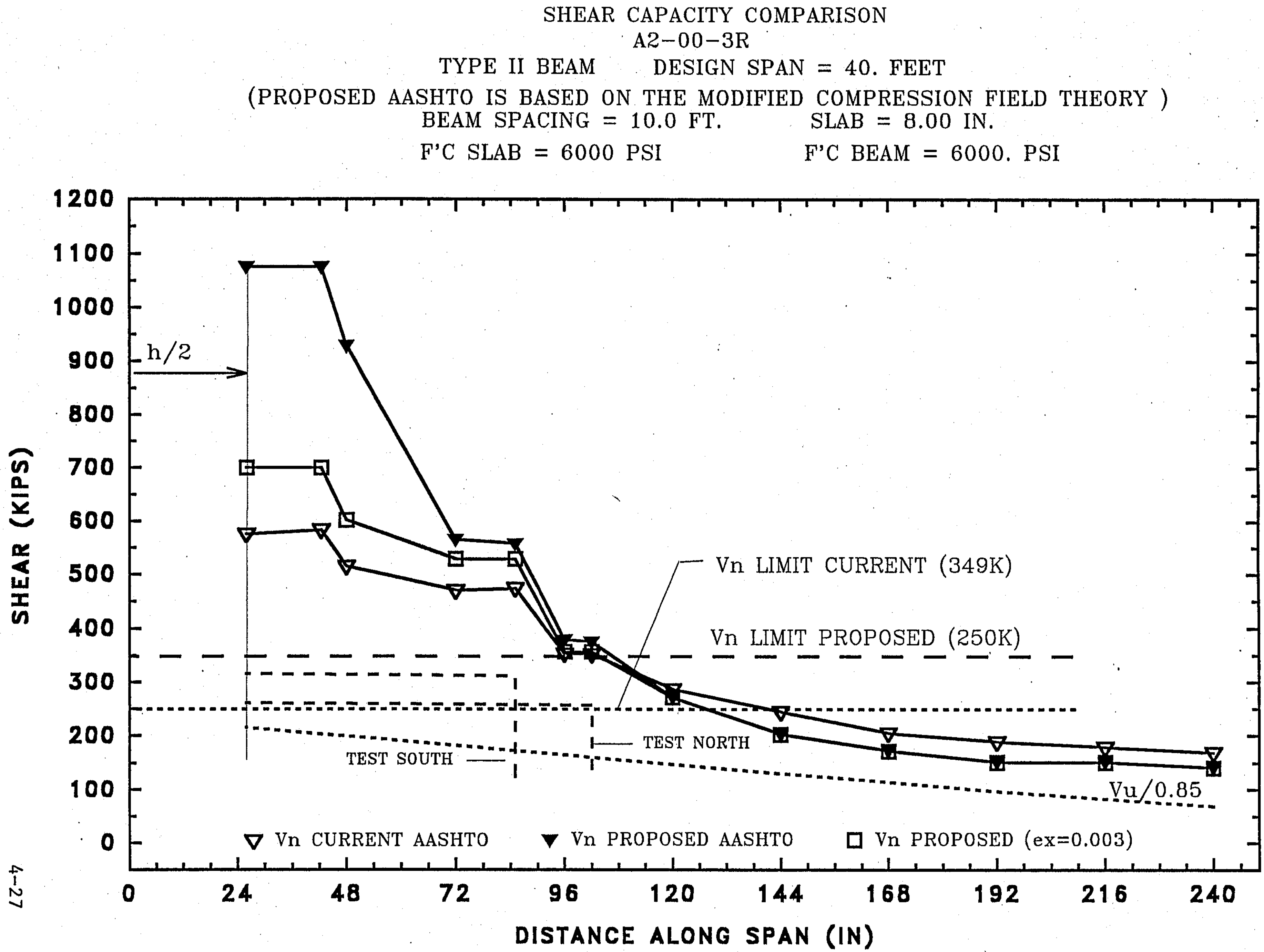

SHEAR CAPACITY COMPARISON
A2-00-3R
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI
SHEAR (KIPS)
1200
1100
1000
900
800
700
600
500
400
300
200
100
0
h/2
Vn LIMIT CURRENT (349K)
Vn LIMIT PROPOSED (250K)
TEST SOUTH
TEST NORTH
Vu/0.85
Vn CURRENT AASHTO
Vn PROPOSED AASHTO
Vn PROPOSED (ex=0.003)
0
24
48
72
96
120
144
168
192
216
240
DISTANCE ALONG SPAN (IN)

FIGURE 4-9

SHEAR CAPACITY COMPARISON
A2-00-3RD
TYPE II BEAM   DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT.   SLAB = 8.00 IN.
F'C SLAB = 6000 PSI   F'C BEAM = 6000. PSI

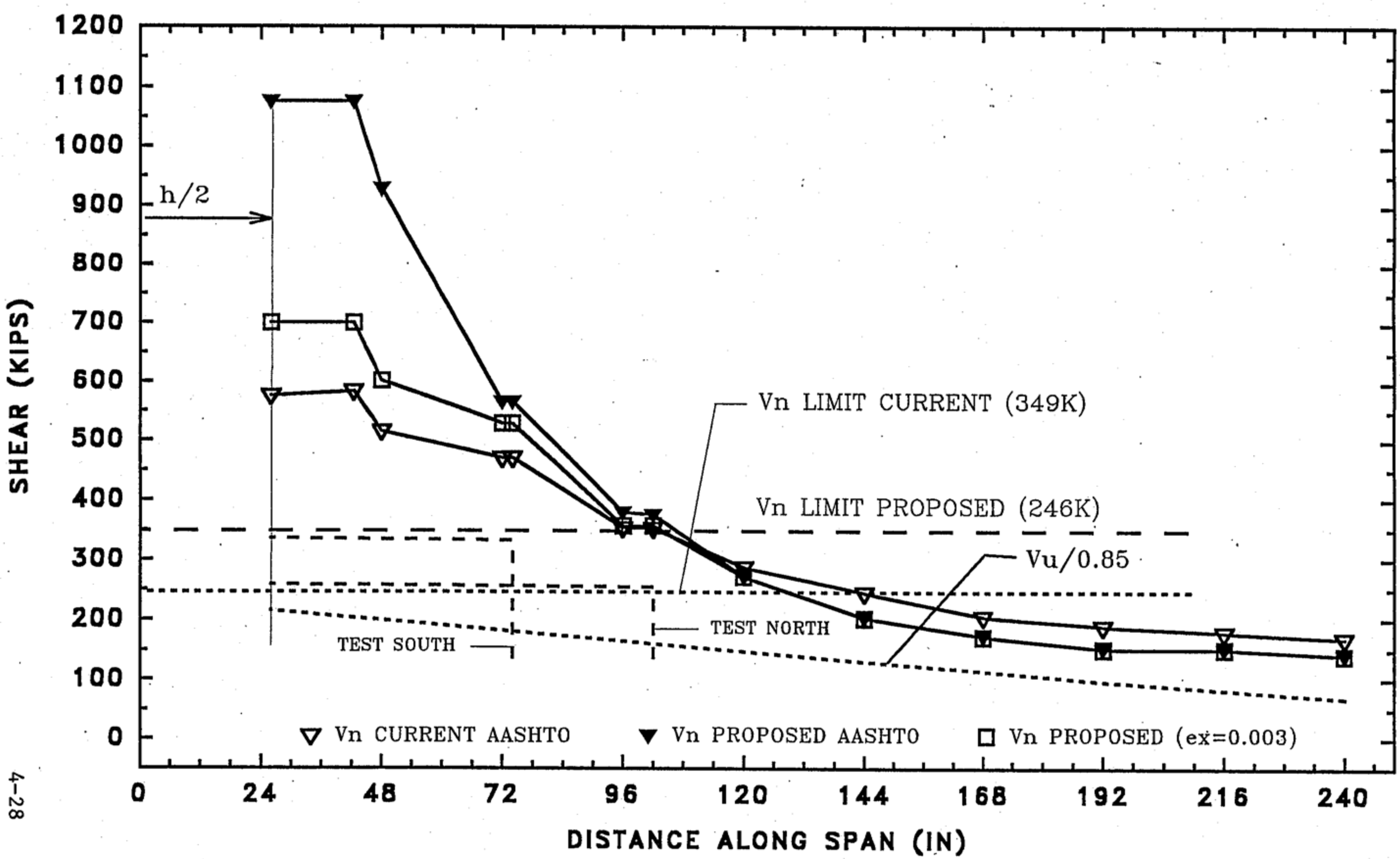


FIGURE 4-10

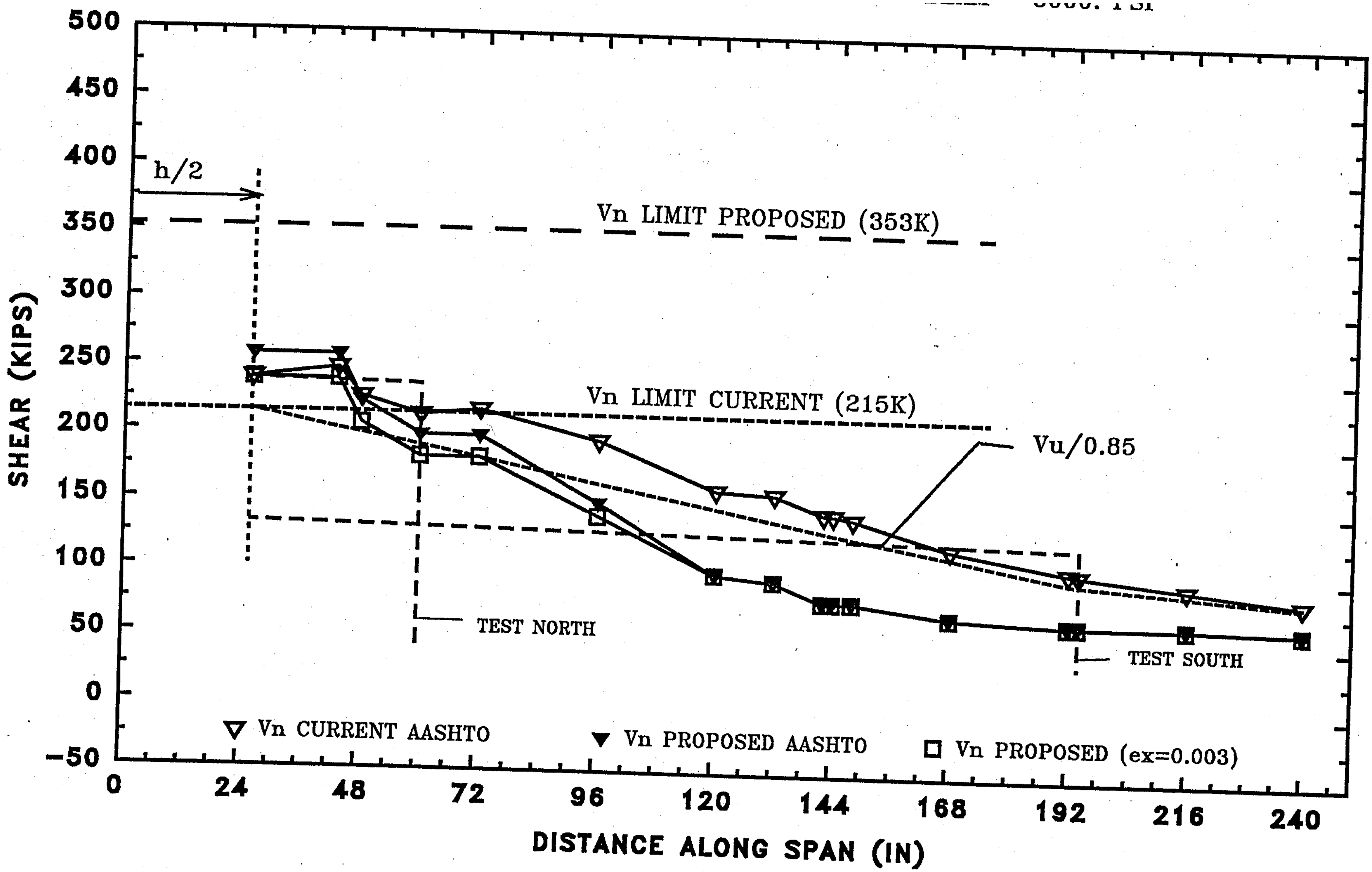

SHEAR (KIPS)
500
450
400
350
300
250
200
150
100
50
0
-50
h/2
Vn LIMIT PROPOSED (353K)
Vn LIMIT CURRENT (215K)
Vu/0.85
TEST NORTH
TEST SOUTH
Vn CURRENT AASHTO
Vn PROPOSED AASHTO
Vn PROPOSED (ex=0.003)
0
24
48
72
96
120
144
168
192
216
240
DISTANCE ALONG SPAN (IN)

FIGURE 4-11

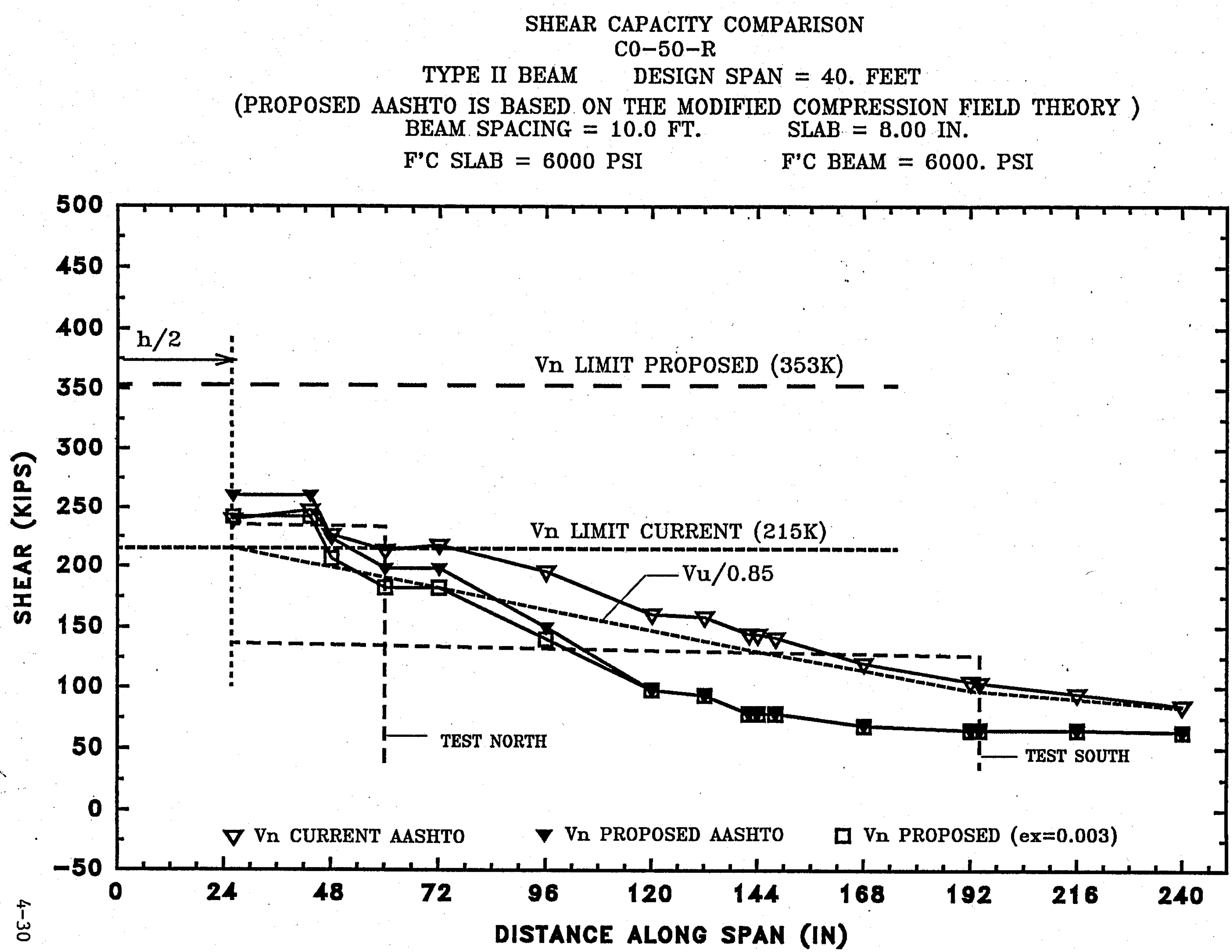

SHEAR CAPACITY COMPARISON
C0-50-R
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI
h/2
Vn LIMIT PROPOSED (353K)
Vn LIMIT CURRENT (215K)
Vu/0.85
TEST NORTH
TEST SOUTH
Vn CURRENT AASHTO
Vn PROPOSED AASHTO
Vn PROPOSED (ex=0.003)
SHEAR (KIPS)
500
450
400
350
300
250
200
150
100
50
0
-50
0
24
48
72
96
120
144
168
192
216
240
DISTANCE ALONG SPAN (IN)

FIGURE 4-12

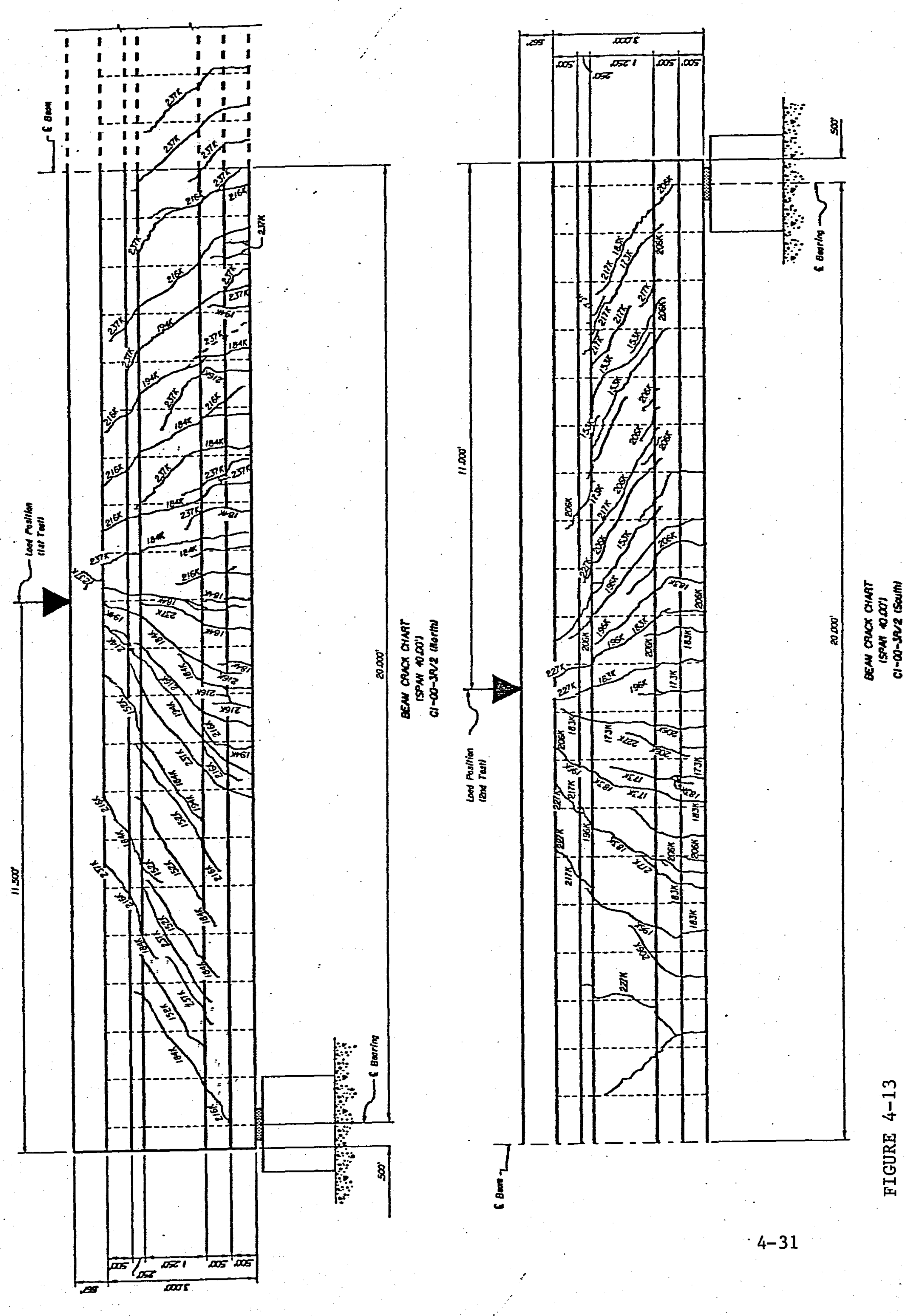

BEAM CRACK CHART
(SPAN 40.00')
CI-00-3R/2 (North)
Load Position (1st Test)
€ Beam
€ Bearing
11.500'
20.000'
3.000'
BEAM CRACK CHART
(SPAN 40.00')
CI-00-3R/2 (South)
Load Position (2nd Test)
11.000'
20.000'
.500'

FIGURE 4-13

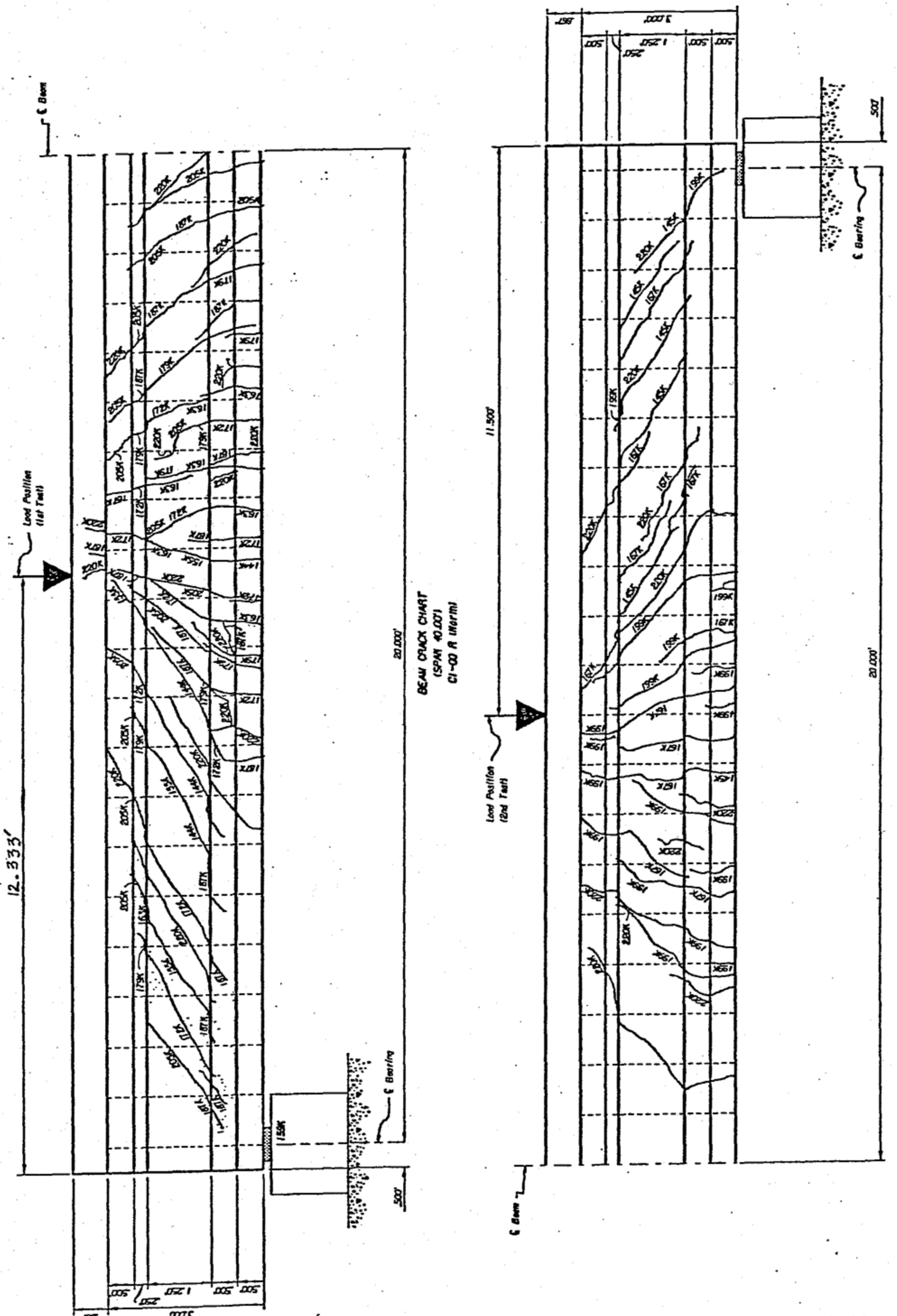

BEAM CRACK CHART
(SPAN 40.00')
C1-00 R (North)
Load Position (1st Test)
12.333'
20.000'
3.000'
BEAM CRACK CHART
(SPAN 40.00')
Load Position (2nd Test)
11.500'
20.000'
3.000'

FIGURE 4-14

SHEAR CAPACITY COMPARISON
A1-00-R/2
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

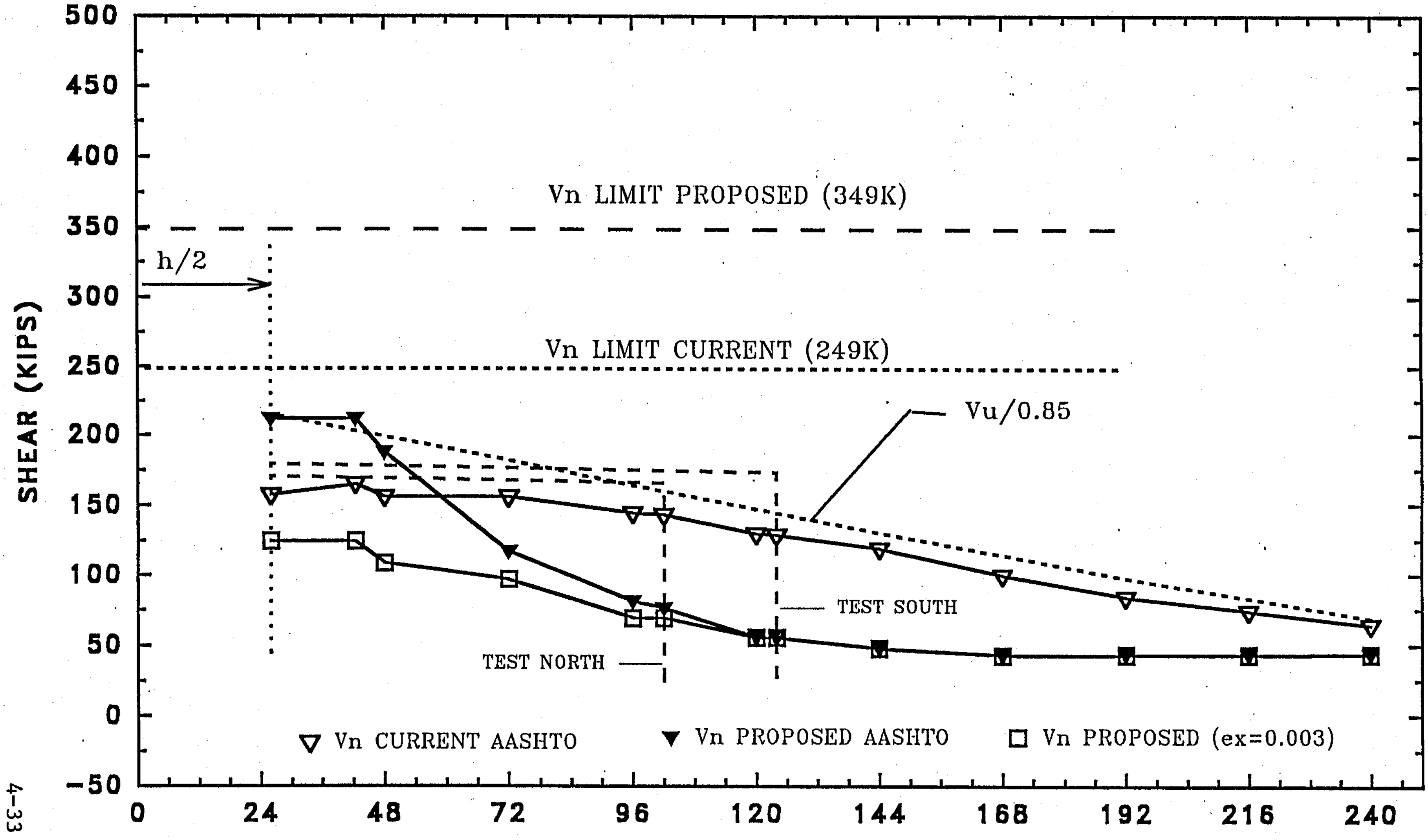


FIGURE 4-15

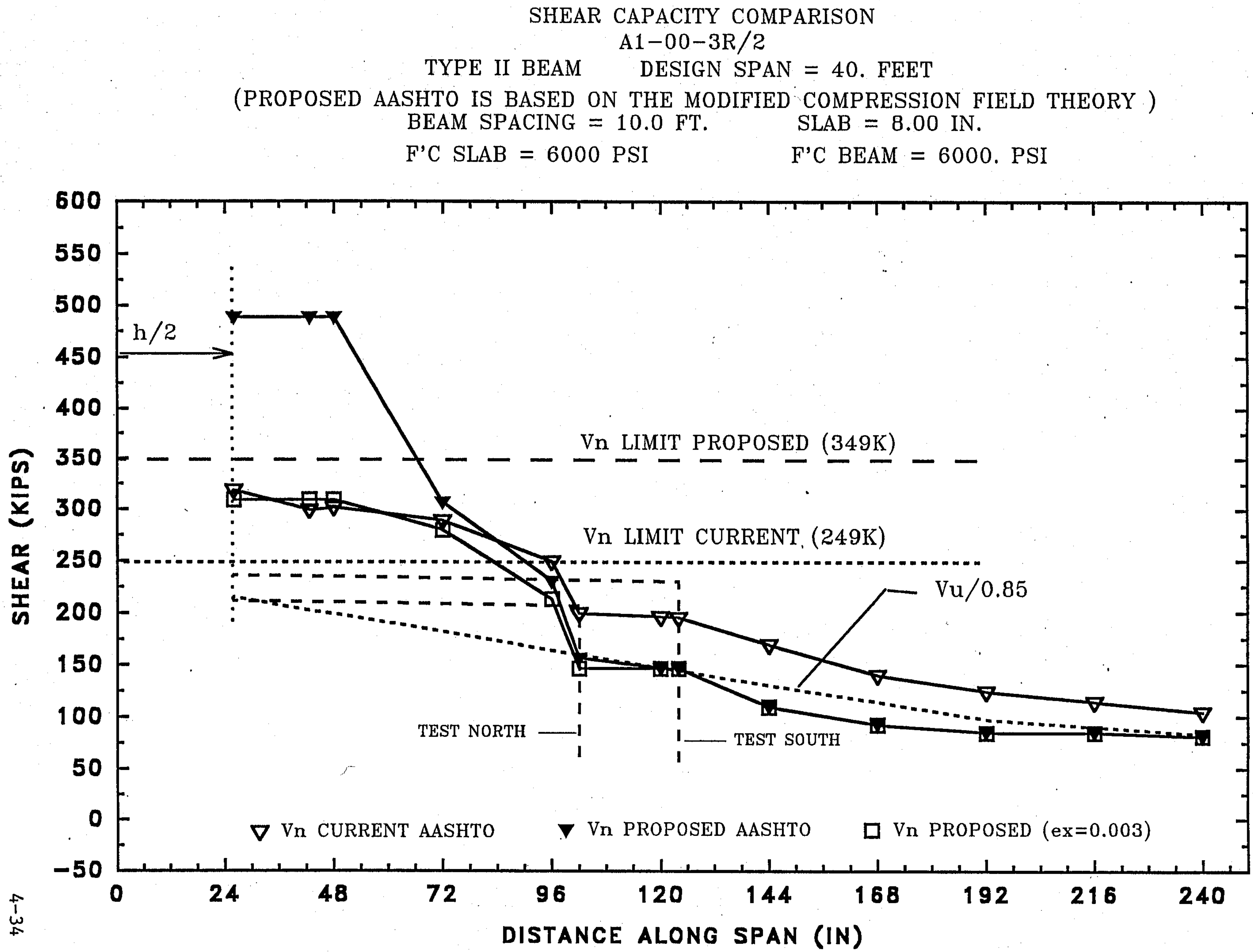

SHEAR CAPACITY COMPARISON
A1-00-3R/2
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI
SHEAR (KIPS)
600
550
500
450
400
350
300
250
200
150
100
50
0
-50
0
24
48
72
96
120
144
168
192
216
240
DISTANCE ALONG SPAN (IN)
h/2
Vn LIMIT PROPOSED (349K)
Vn LIMIT CURRENT (249K)
Vu/0.85
TEST NORTH
TEST SOUTH
Vn CURRENT AASHTO
Vn PROPOSED AASHTO
Vn PROPOSED (ex=0.003)

FIGURE 4-16

SHEAR CAPACITY COMPARISON
A-SERIES GIRDERS
TYPE II BEAM   DESIGN SPAN = 40. FEET

BEAM SPACING = 10.0 FT.   SLAB = 8.00 IN.
F'C SLAB = 6000 PSI   F'C BEAM = 6000. PSI

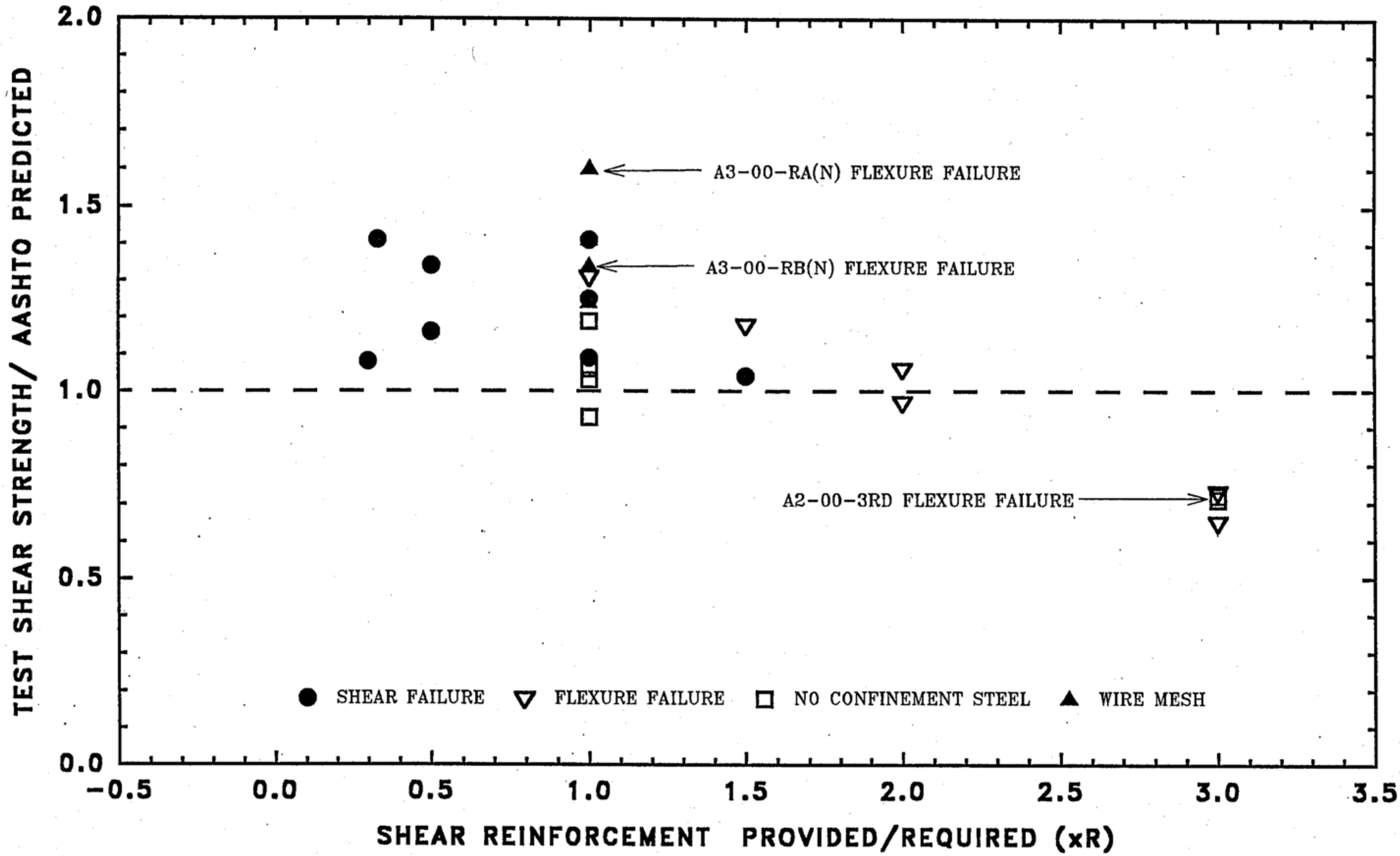


FIGURE 4-17

SHEAR CAPACITY COMPARISON
A2-00-2R
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

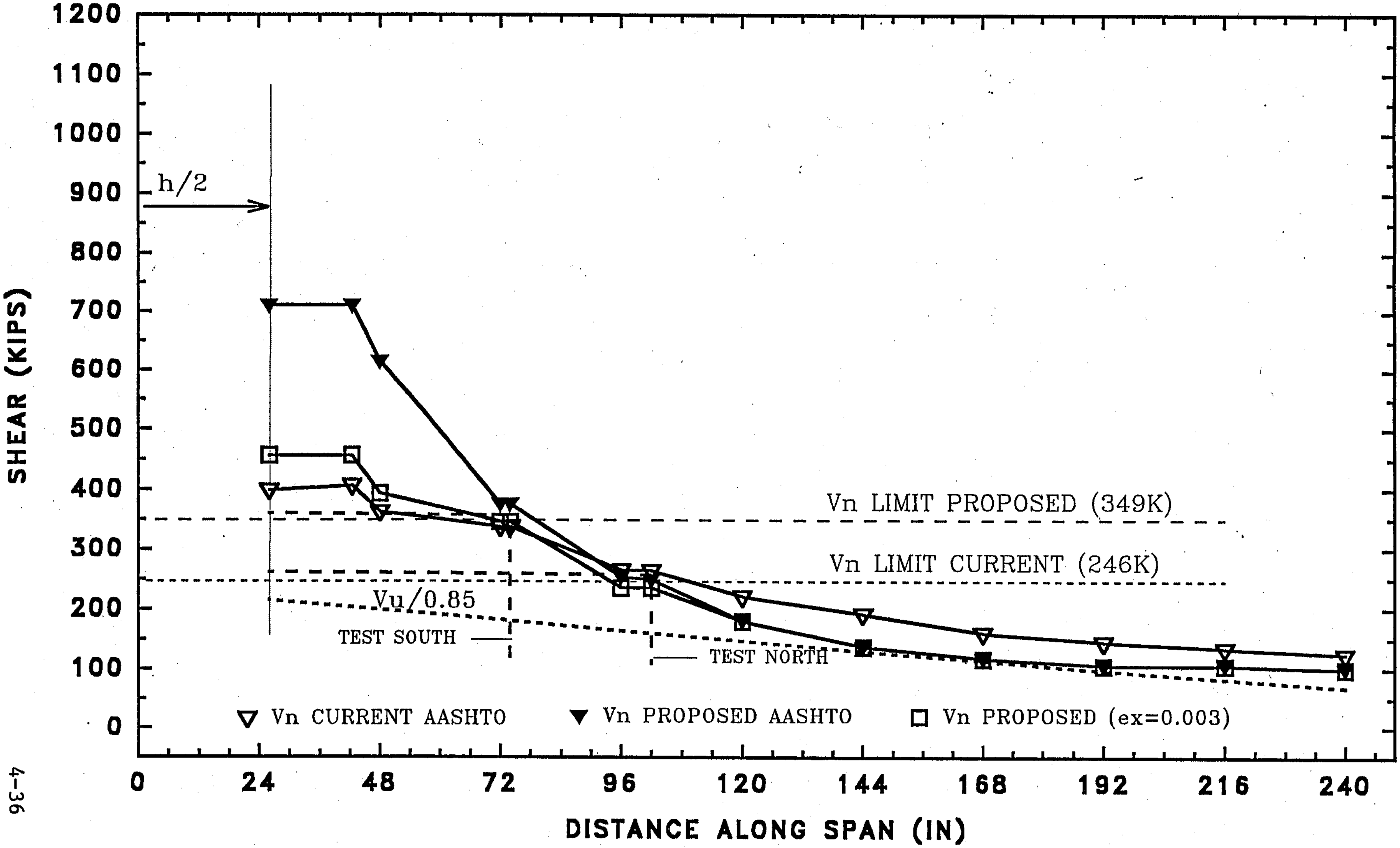


FIGURE 4-18

SHEAR CAPACITY COMPARISON
B0-00-R
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

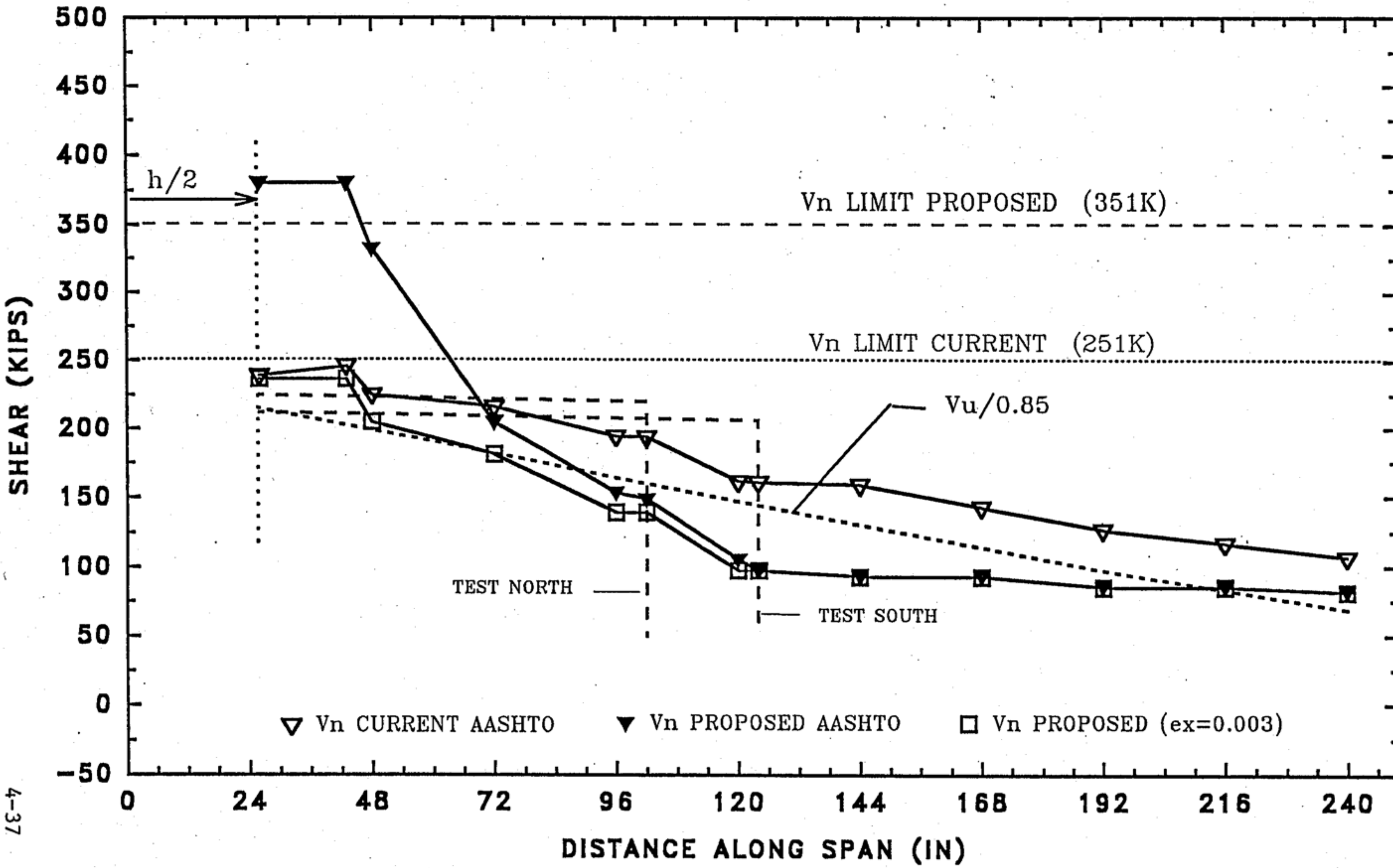


FIGURE 4-19

SHEAR CAPACITY COMPARISON
B0-00-2R
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

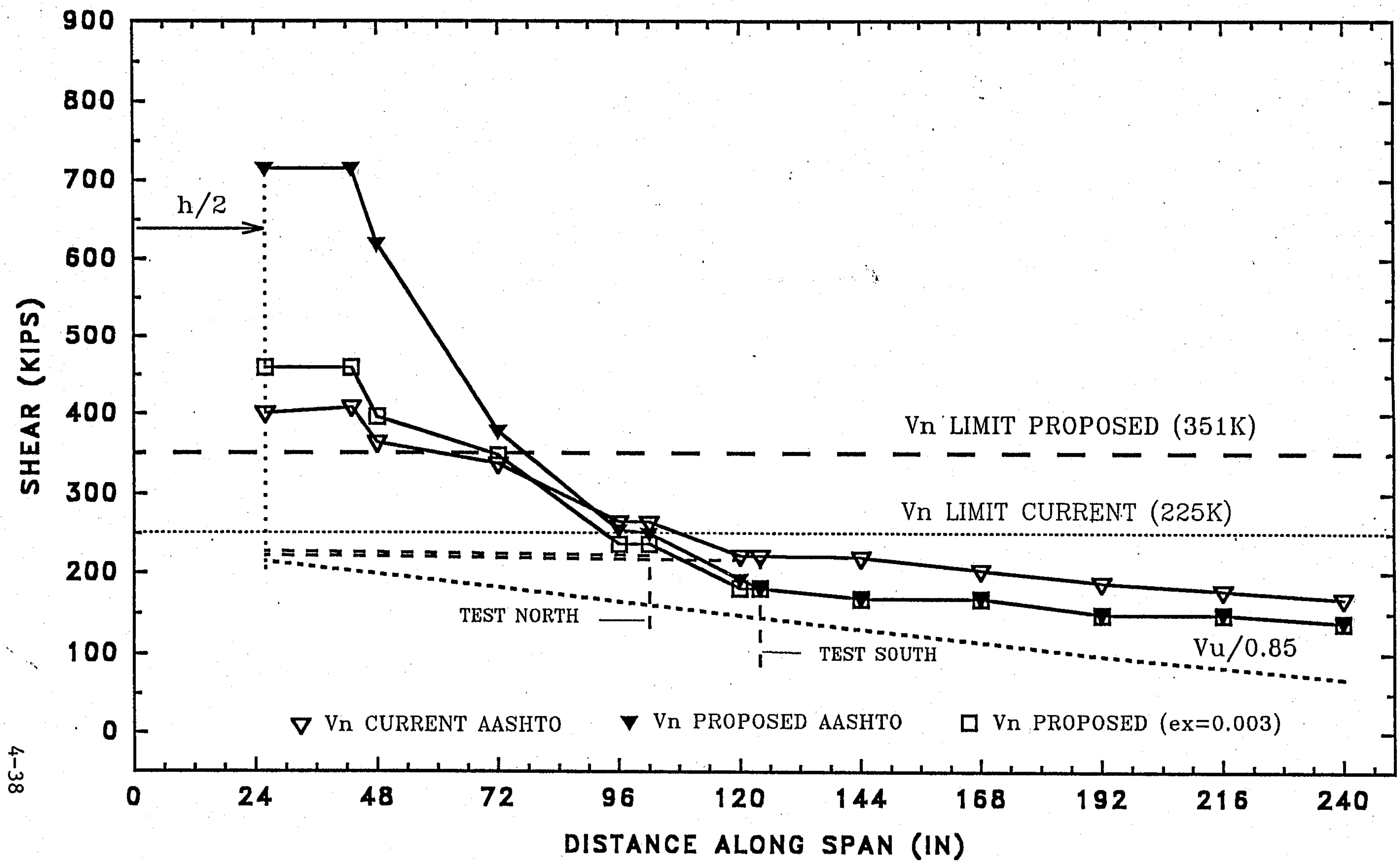


FIGURE 4-20

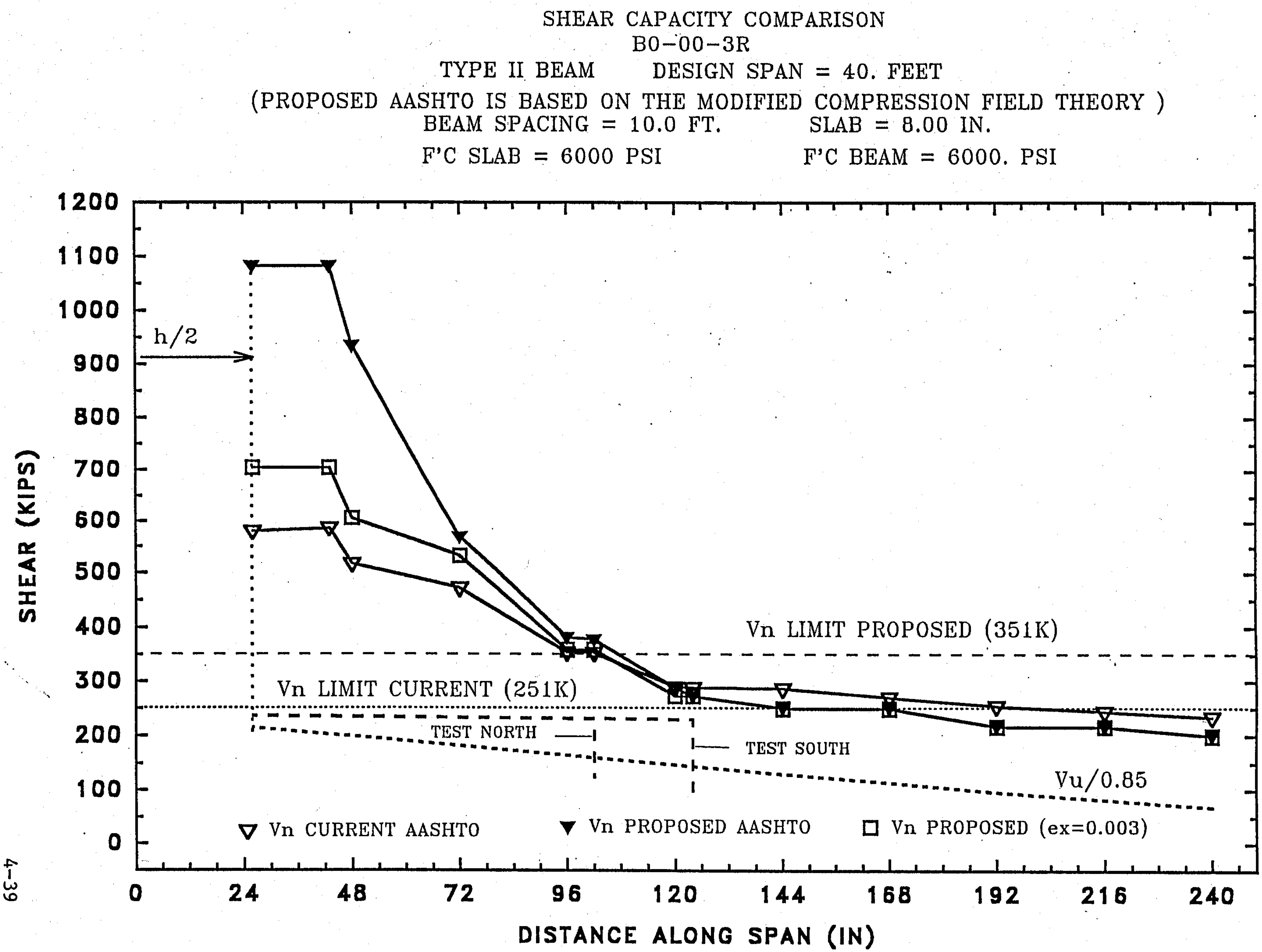
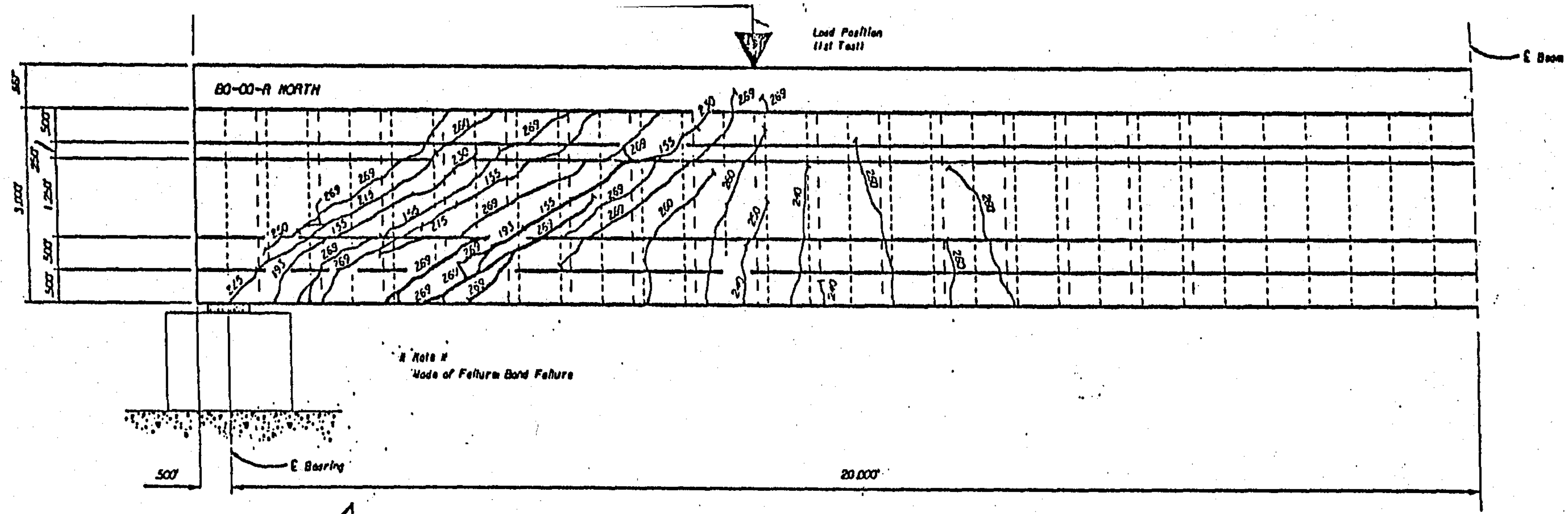

SHEAR CAPACITY COMPARISON
B0-00-3R
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI
SHEAR (KIPS)
1200
1100
1000
900
800
700
600
500
400
300
200
100
0
h/2
Vn LIMIT PROPOSED (351K)
Vn LIMIT CURRENT (251K)
TEST NORTH
TEST SOUTH
Vu/0.85
Vn CURRENT AASHTO
Vn PROPOSED AASHTO
Vn PROPOSED (ex=0.003)
0
24
48
72
96
120
144
168
192
216
240
DISTANCE ALONG SPAN (IN)

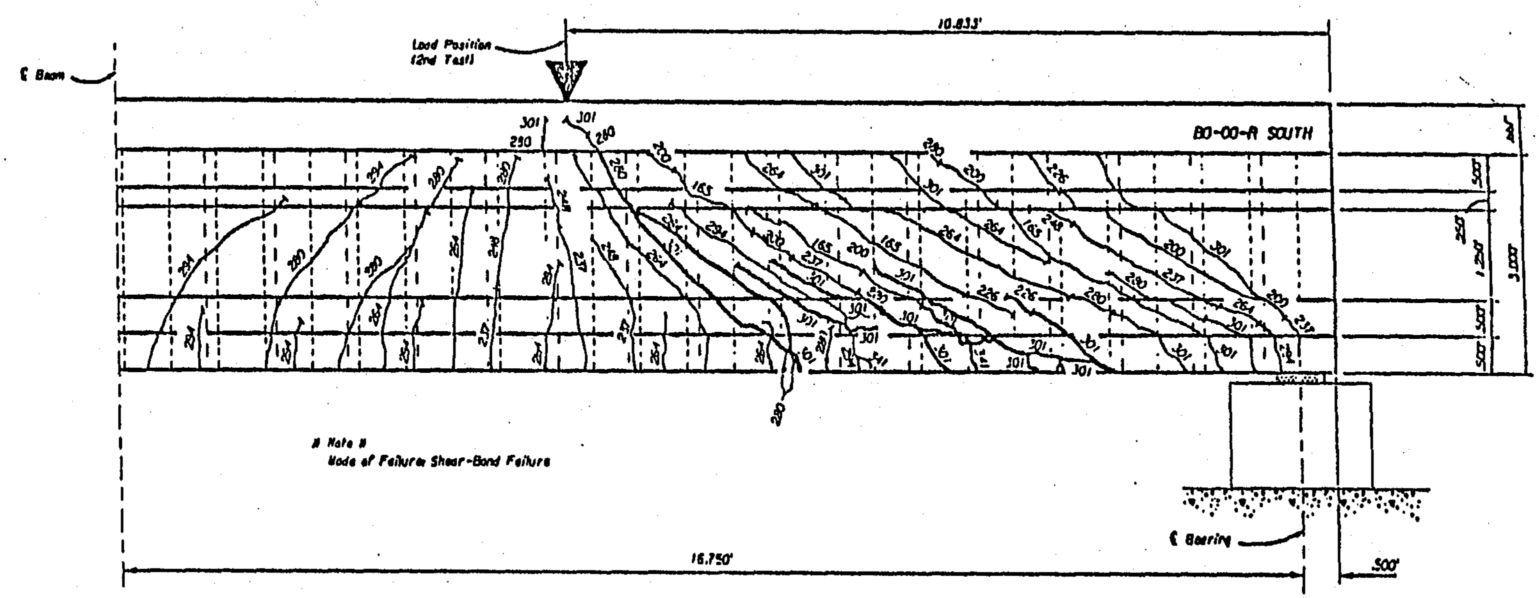
FIGURE 4-21



BEAM CRACK CHART
BEAM BO-OO-R NORTH
(SPAN 40.00')

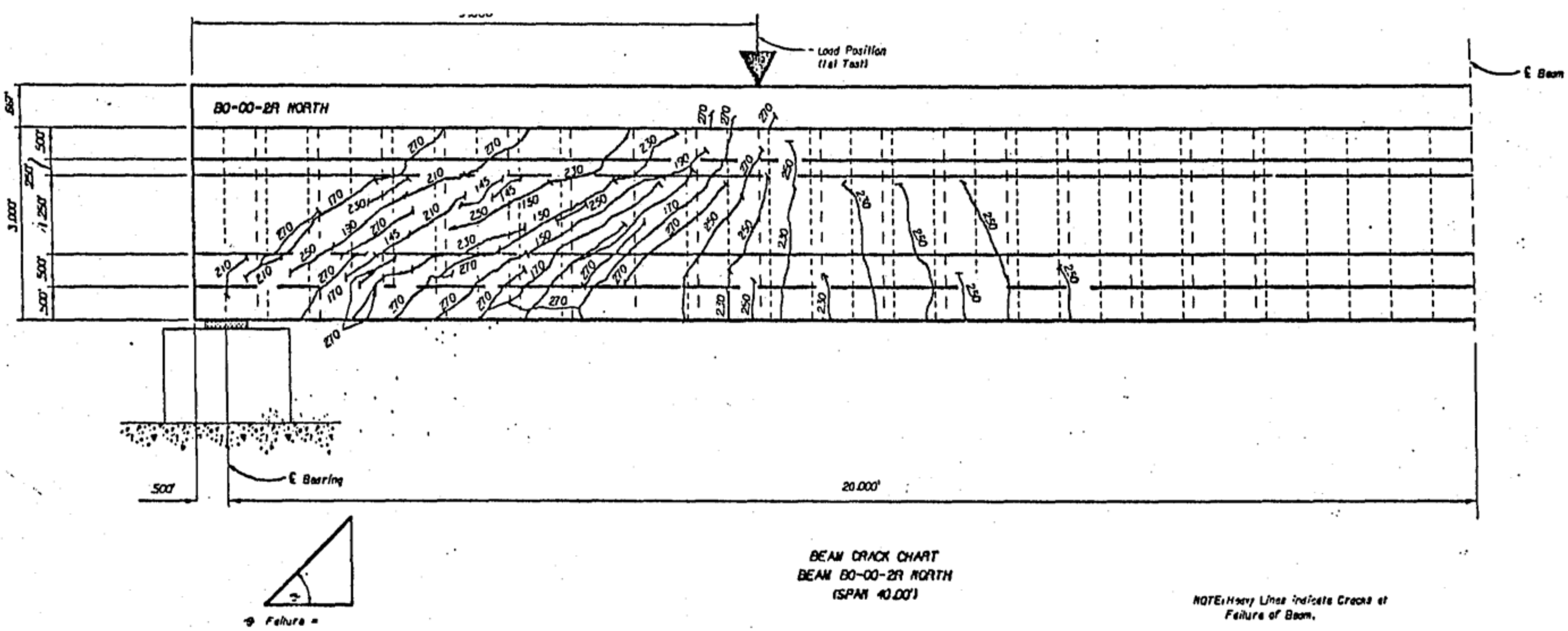
-θ Failure =

NOTE: Heavy Lines Indicate Cracks at Failure of Beam.



BEAM CRACK CHART
BEAM BO-OO-R SOUTH
(SPAN 31.50')

FIGURE 4-22



BEAM CRACK CHART
BEAM BO-00-2R NORTH
(SPAN 40.00')

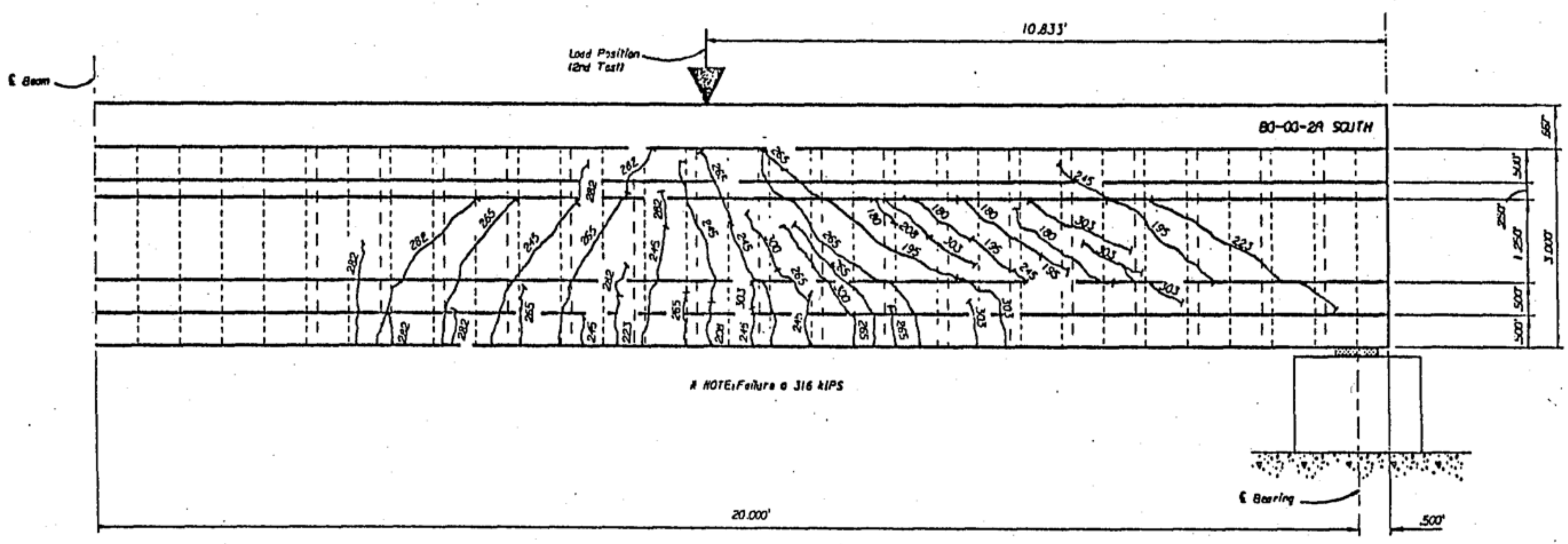
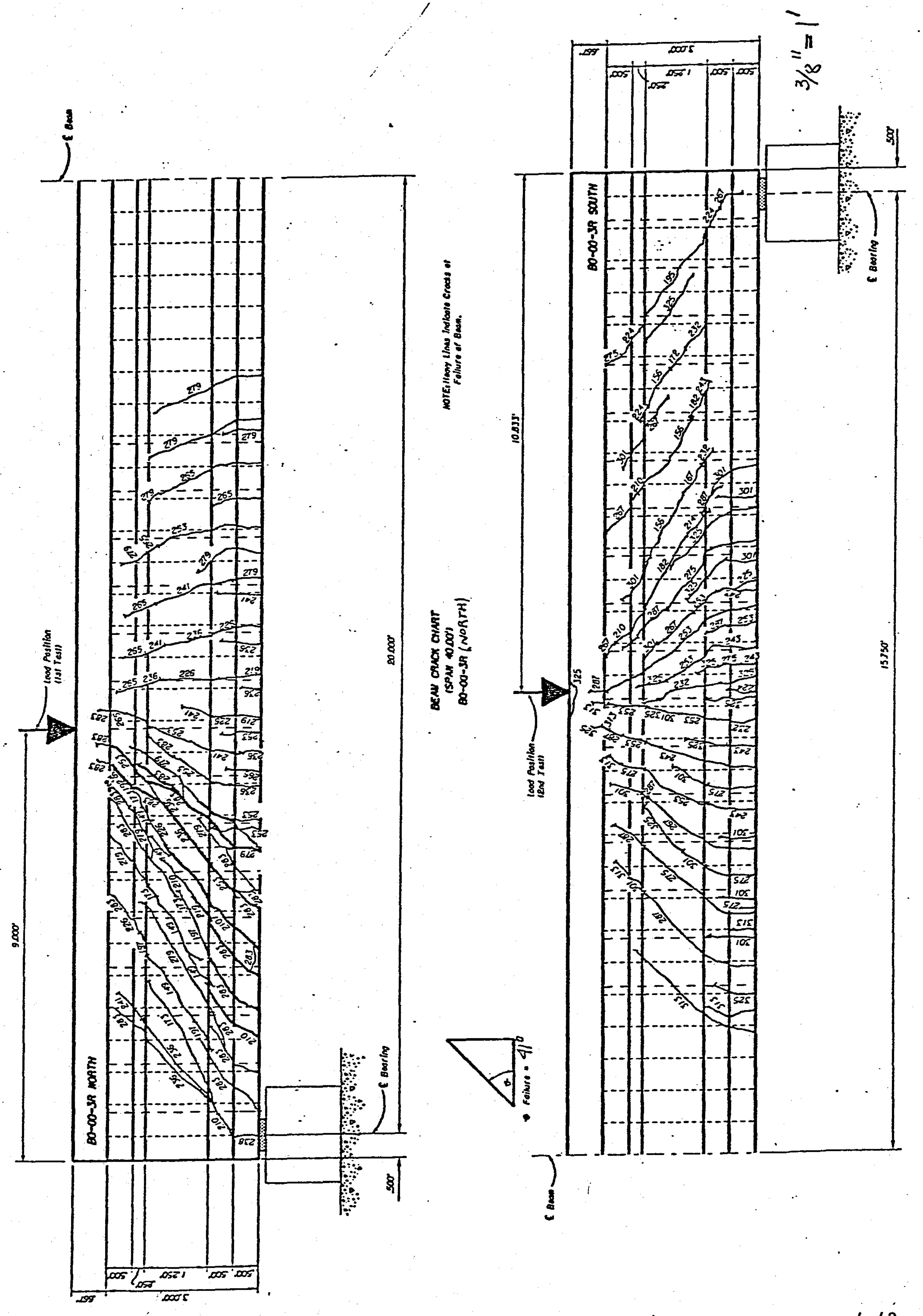


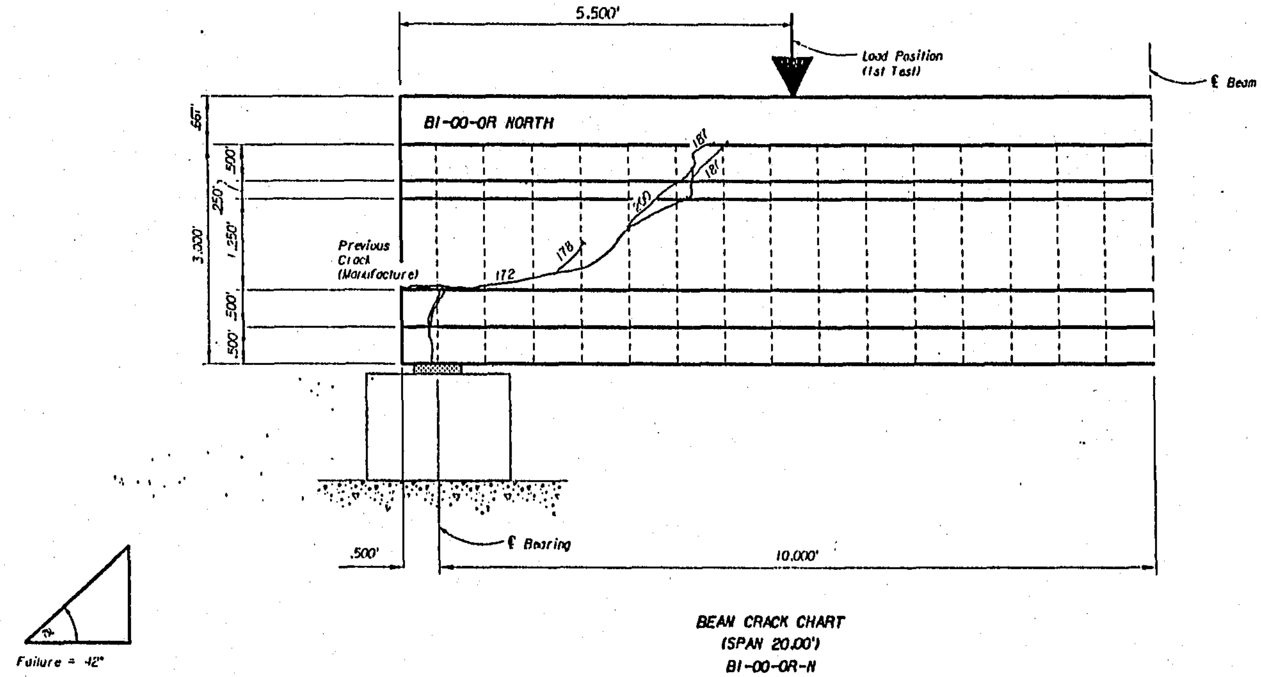
FIGURE 4-23

BEAM CRACK CHART
BEAM BO-00-2R SOUTH
(SPAN 31.50')



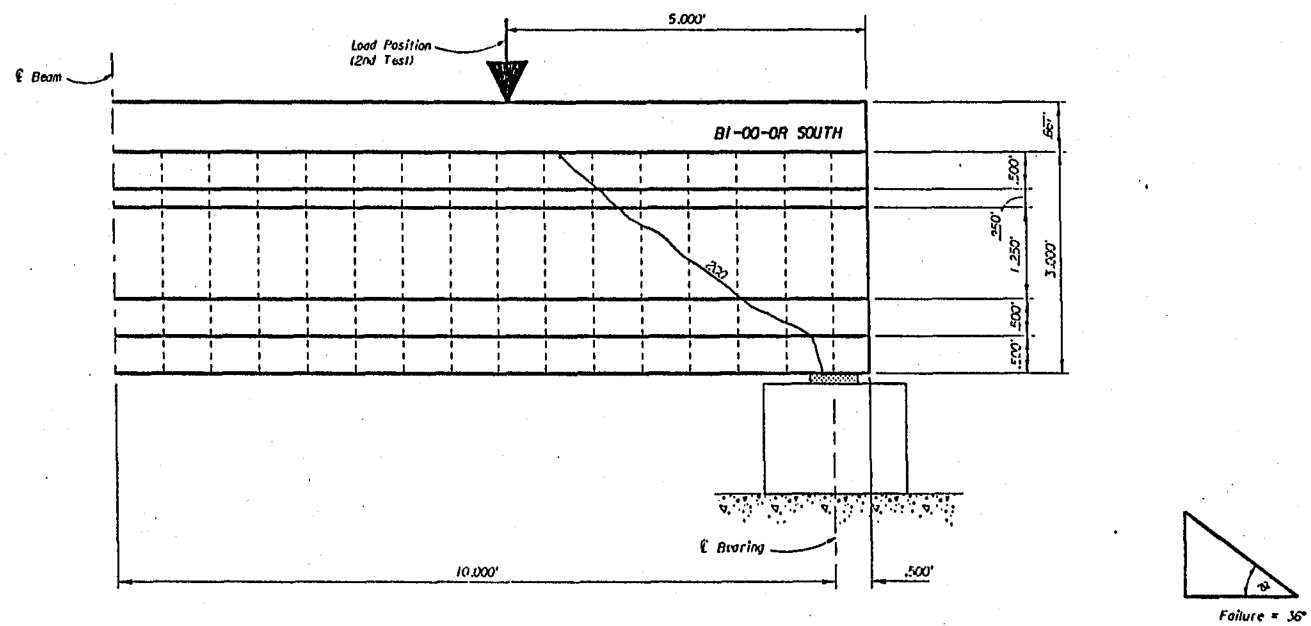
BEAM CRACK CHART
(SPAN 40.00')
BO-00-3R (NORTH)
BEAM CRACK CHART
(SPAN 31.5')
BO-00-3R SOUTH
BO-00-3R NORTH
BO-00-3R SOUTH
NOTE: Heavy Lines Indicate Cracks at Failure of Beam.
Load Position (1st Test)
Load Position (2nd Test)
℄ Beam
℄ Bearing
9.000'
20.000'
10.833'
15.750'
.500'
3/8" = 1'
Failure = 41°

FIGURE 4-24



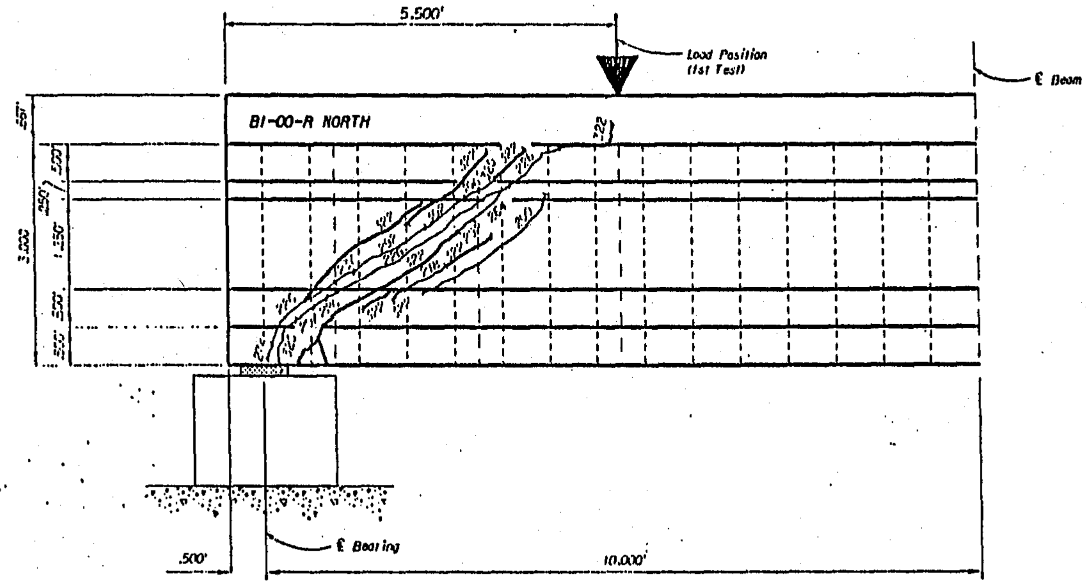
BEAM CRACK CHART
(SPAN 20.00')
BI-00-OR-N

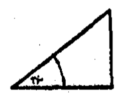
NOTE: Heavy Lines indicate Cracks at Failure of Beam.



BEAM CRACK CHART
(SPAN 18.50')
BI-00-OR-S

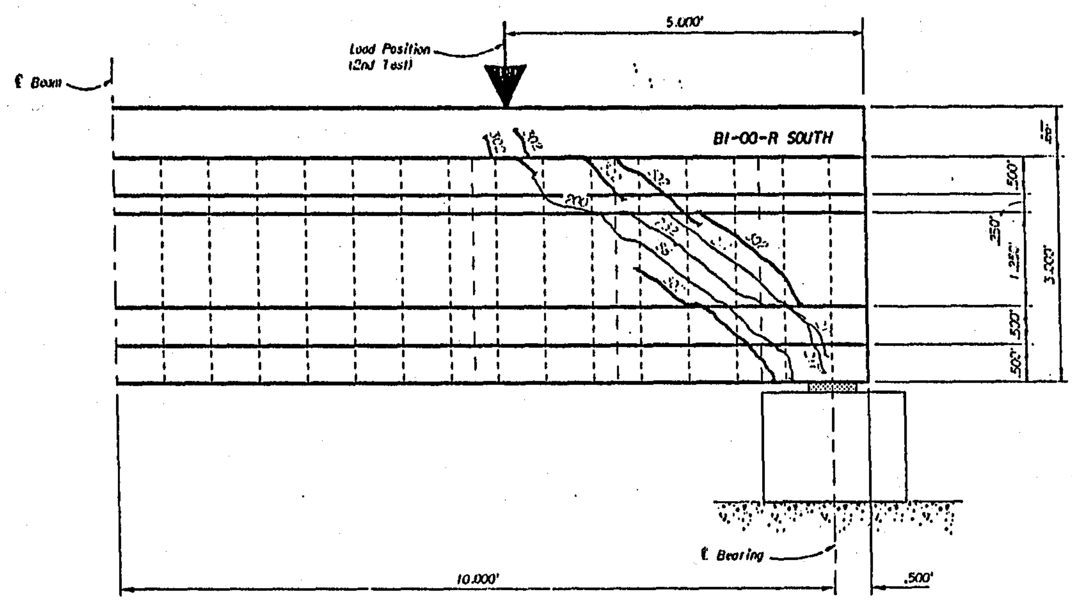
FIGURE 4-25

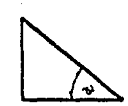




BEAM CRACK CHART
(SPAN 20.00')
BI-OO-R NORTH
TEST DATE : 8-1-92

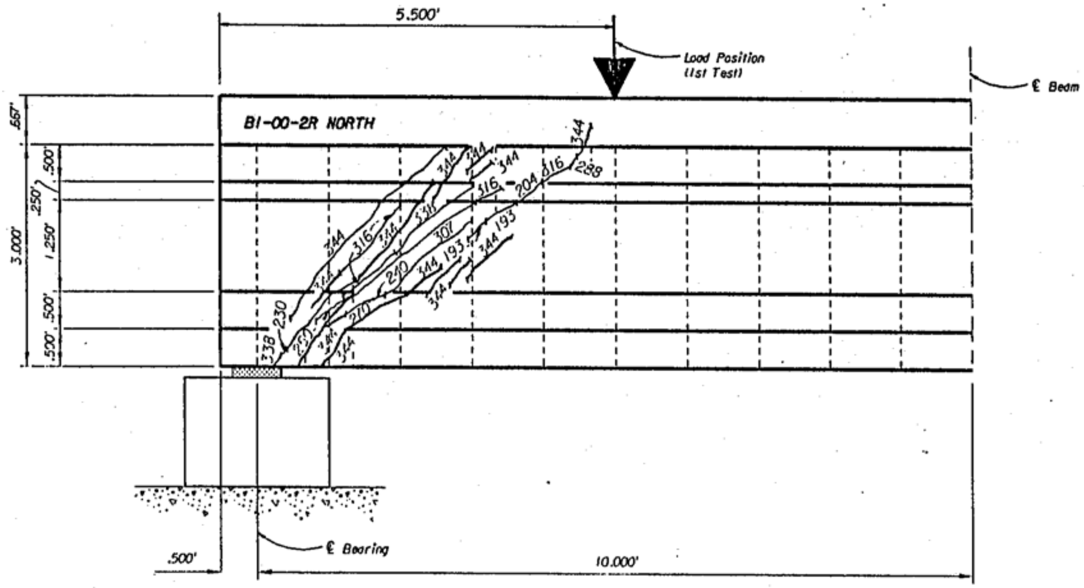
NOTE: Heavy Lines Indicate Cracks at Failure of Beam.





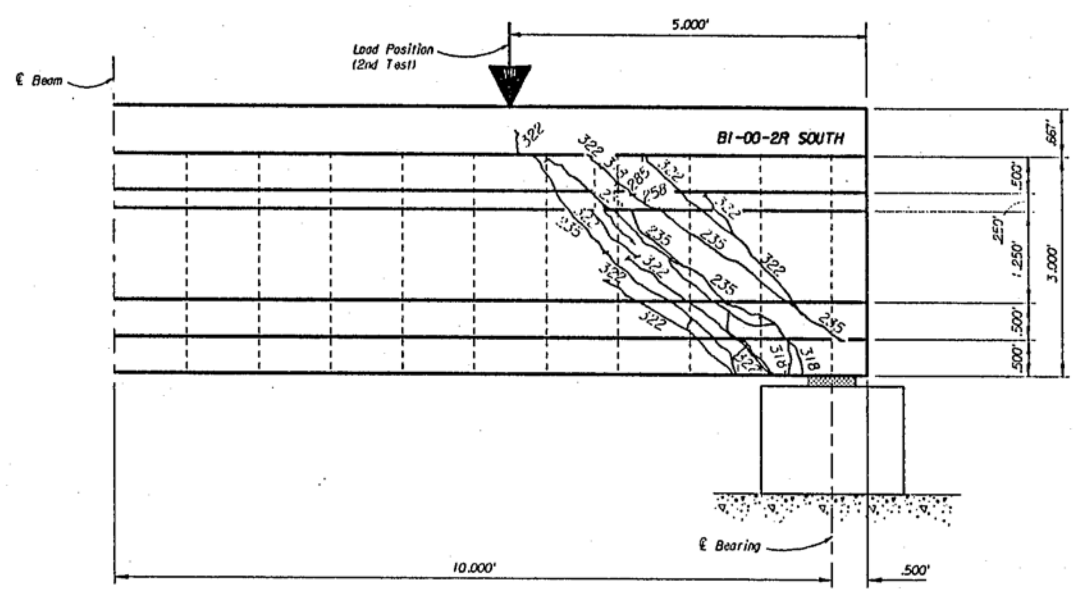
BEAM CRACK CHART
(SPAN 18.50')
BI-OO-R SOUTH
TEST DATE : 8-1-92

FIGURE 4-26



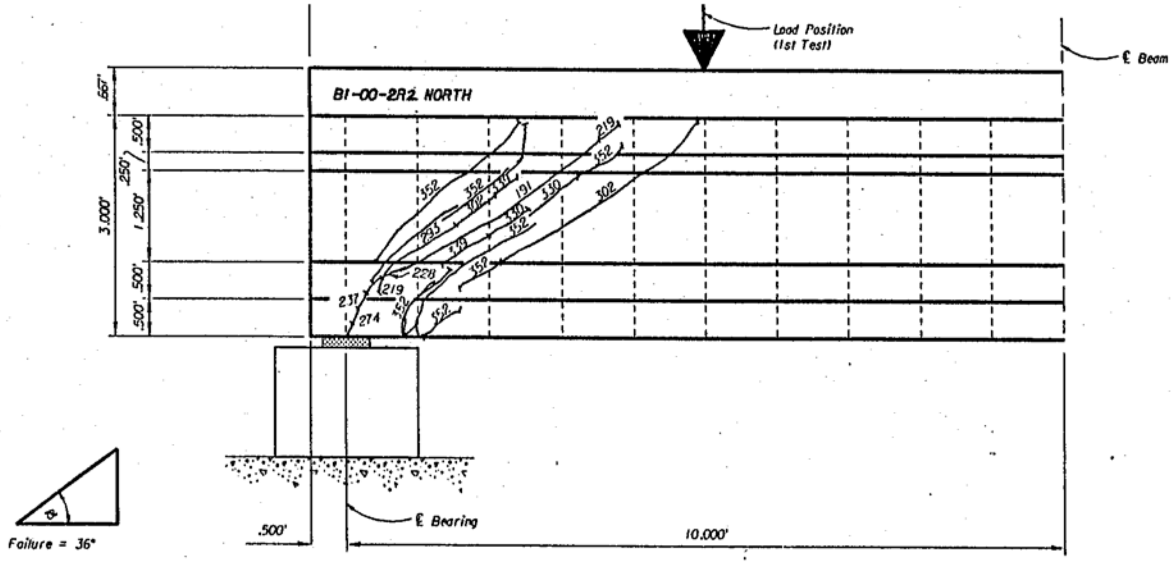
BEAM CRACK CHART
(SPAN 20.00')
BI-OO-2R NORTH

NOTE: Heavy Lines indicate Cracks at Failure of Beam.



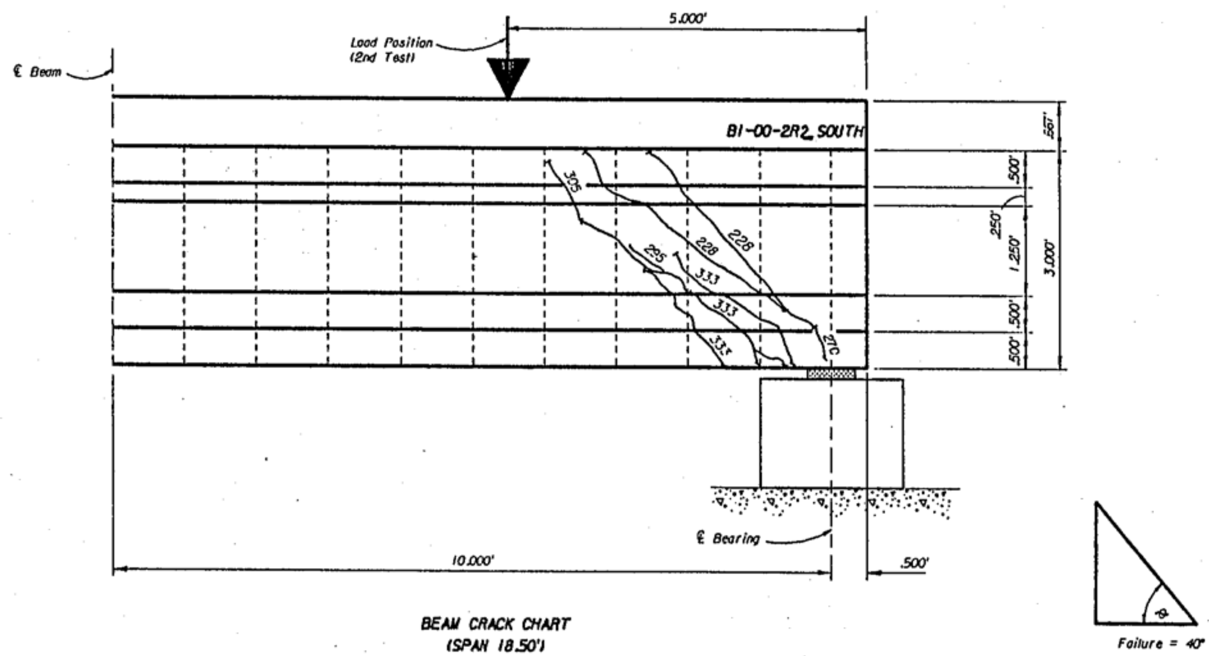
BEAM CRACK CHART
(SPAN 18.50')
BI-OO-2R SOUTH

FIGURE 4-27a



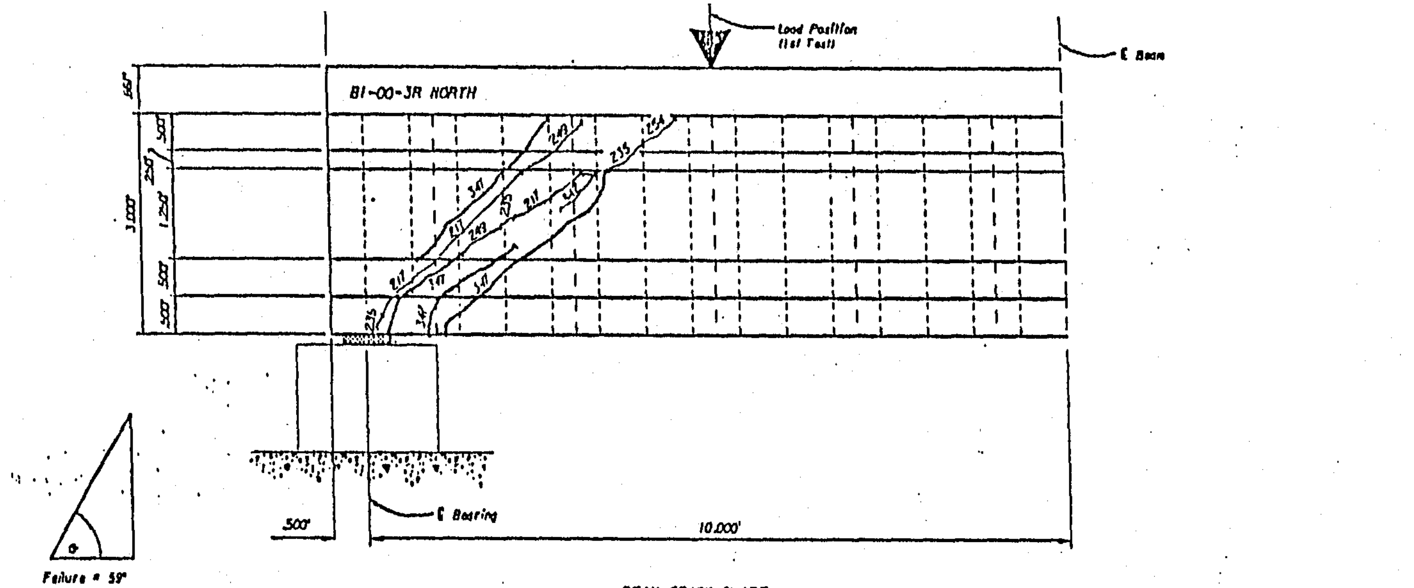
BEAM CRACK CHART
(SPAN 20.00')
BI-00-2R2 NORTH

NOTE: Heavy Lines indicate Cracks at Failure of Beam.

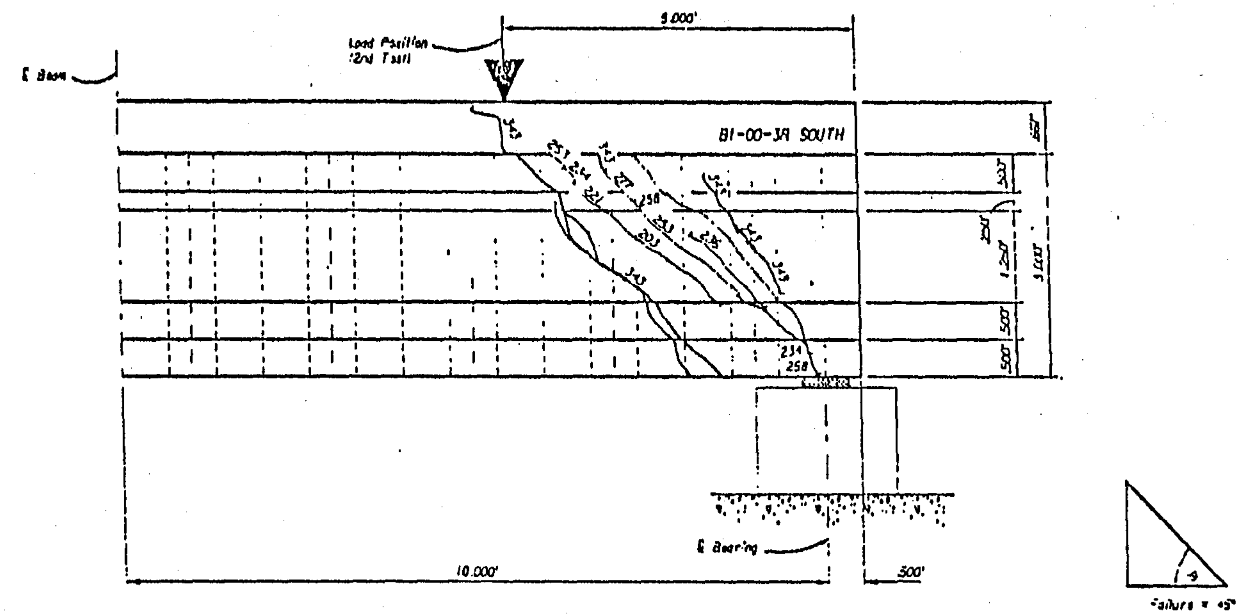


BEAM CRACK CHART
(SPAN 18.50')
BI-00-2R2 SOUTH

FIGURE 4-27b



BEAM CRACK CHART
(SPAN 20.00')
BI-00-3R NORTH
TEST DATE: 8-19-92



BEAM CRACK CHART
(SPAN 18.50')
BI-00-3R SOUTH

FIGURE 4-28

SHEAR CAPACITY COMPARISON
B1-00-0R
TYPE II BEAM DESIGN SPAN = 20. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

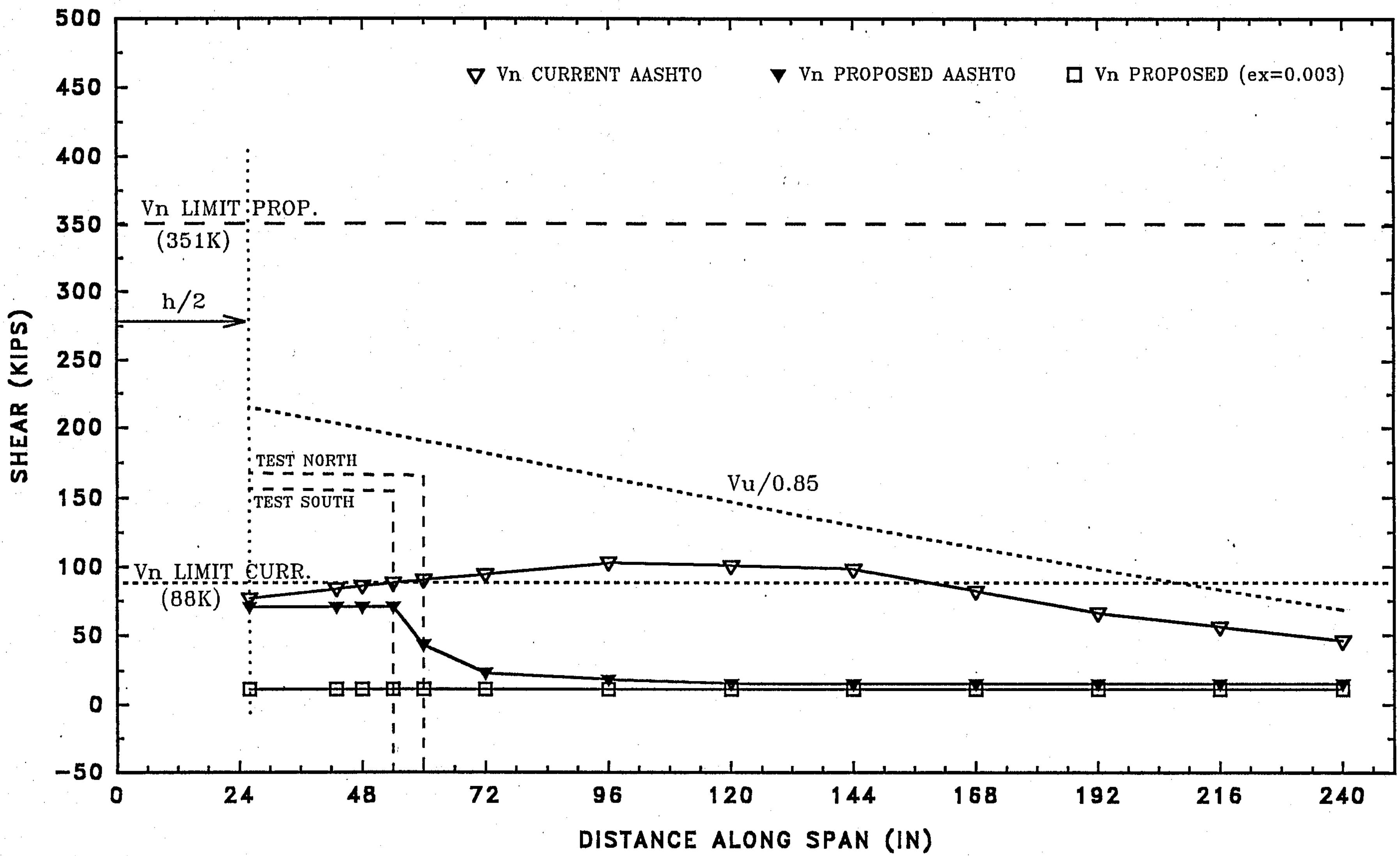


FIGURE 4-29

SHEAR CAPACITY COMPARISON
B1-00-R
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

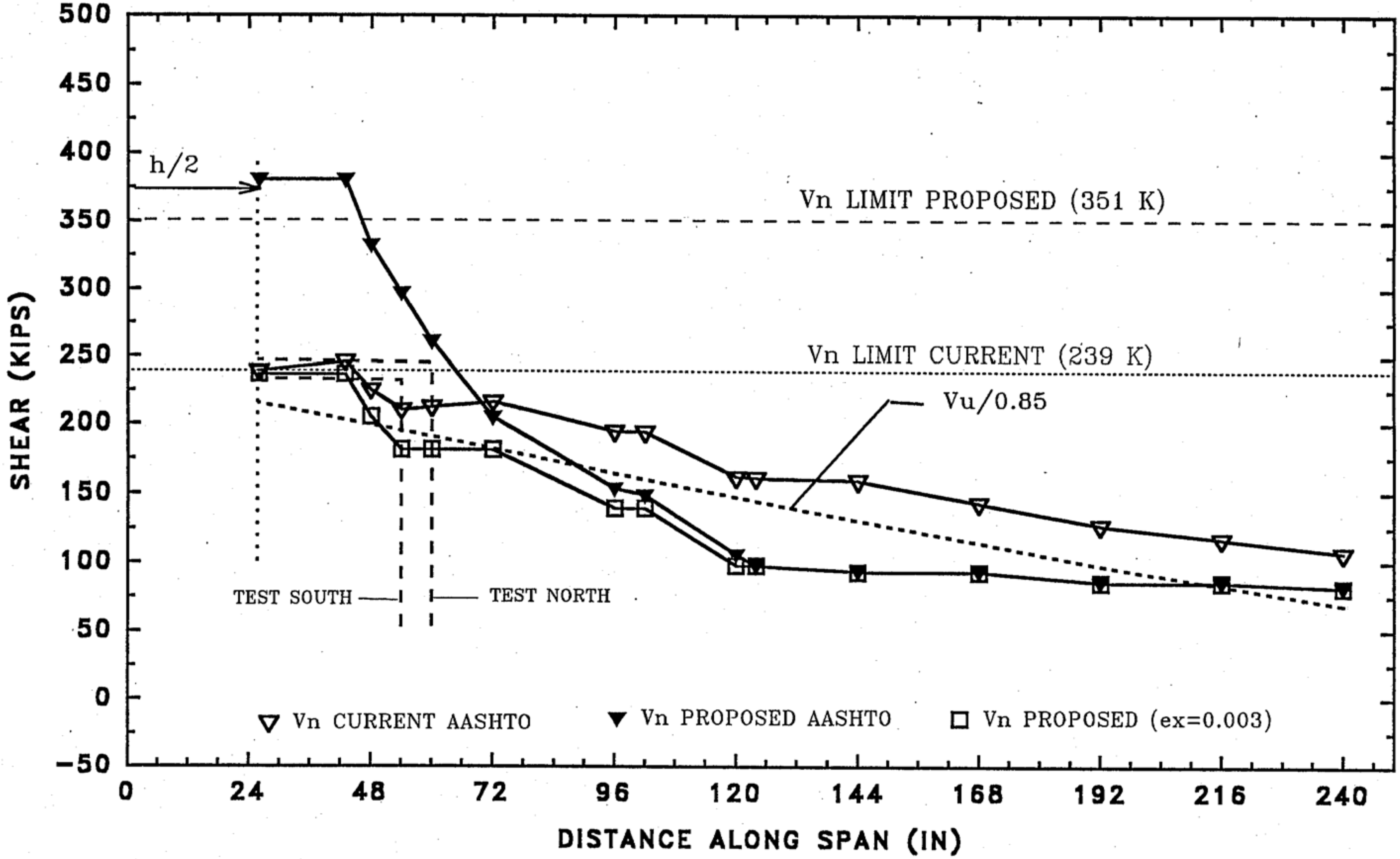


FIGURE 4-30

SHEAR CAPACITY COMPARISON
B1-00-2R
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

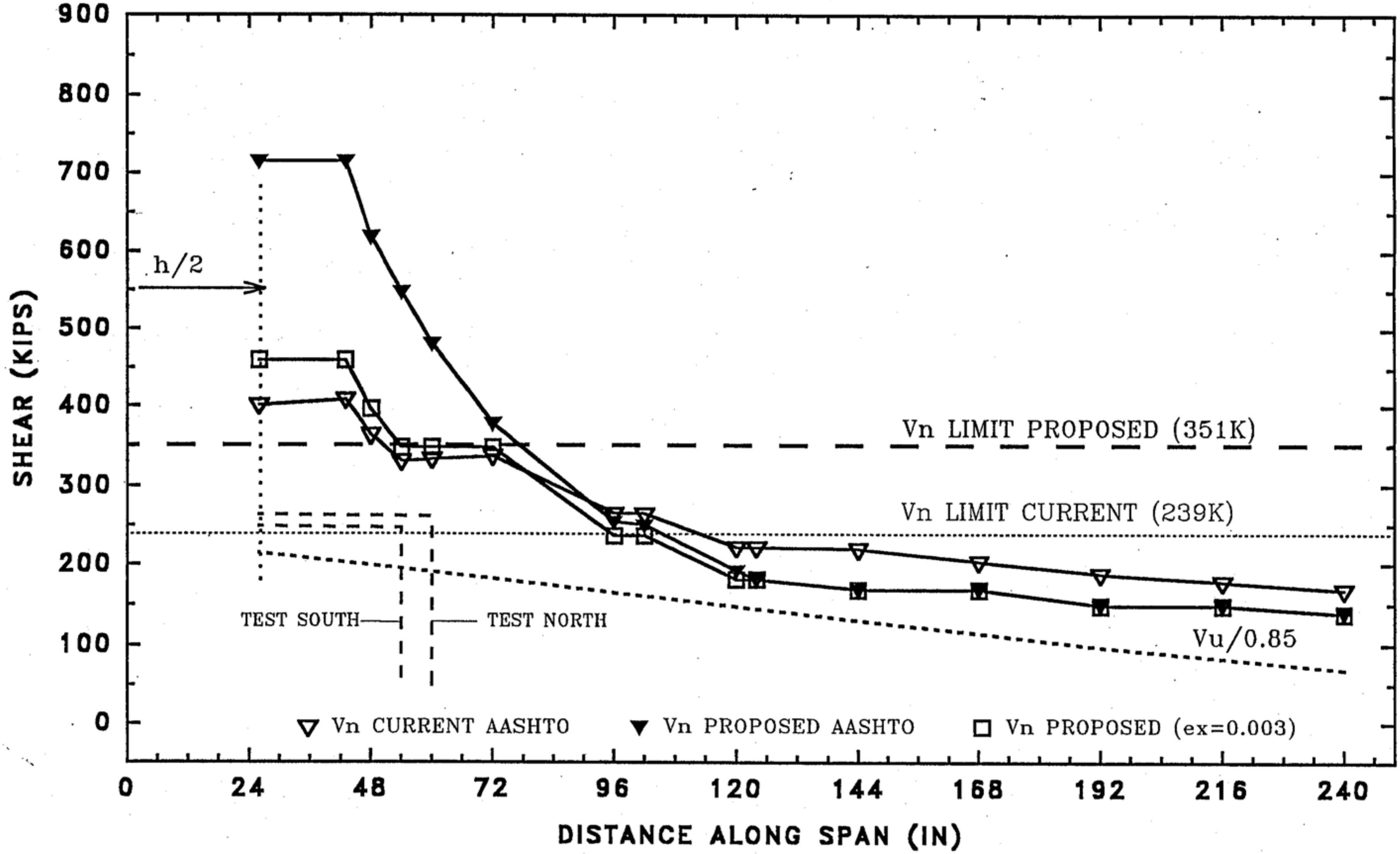


FIGURE 4-31

SHEAR CAPACITY COMPARISON
B1-00-2R #2
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

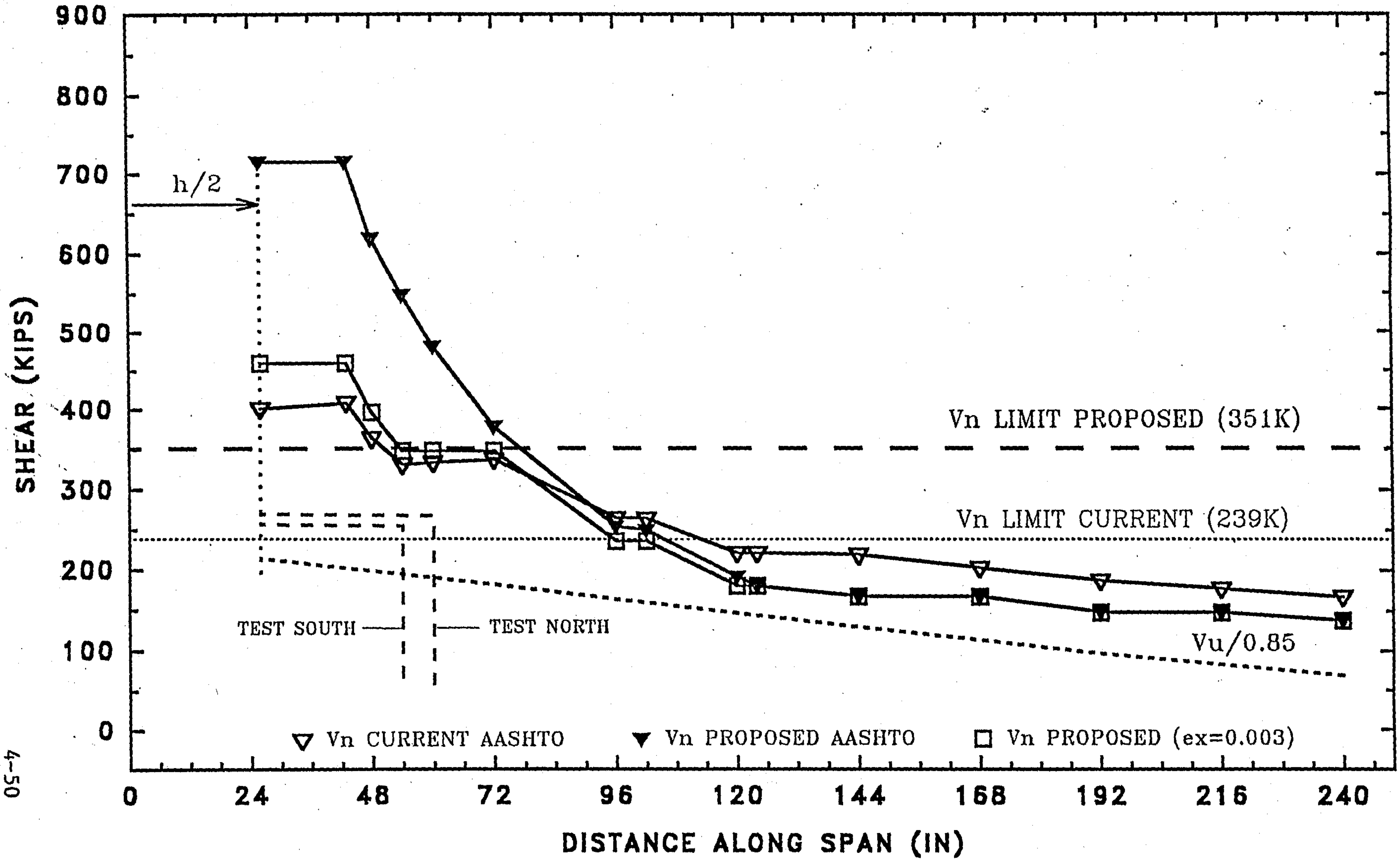


FIGURE 4-32

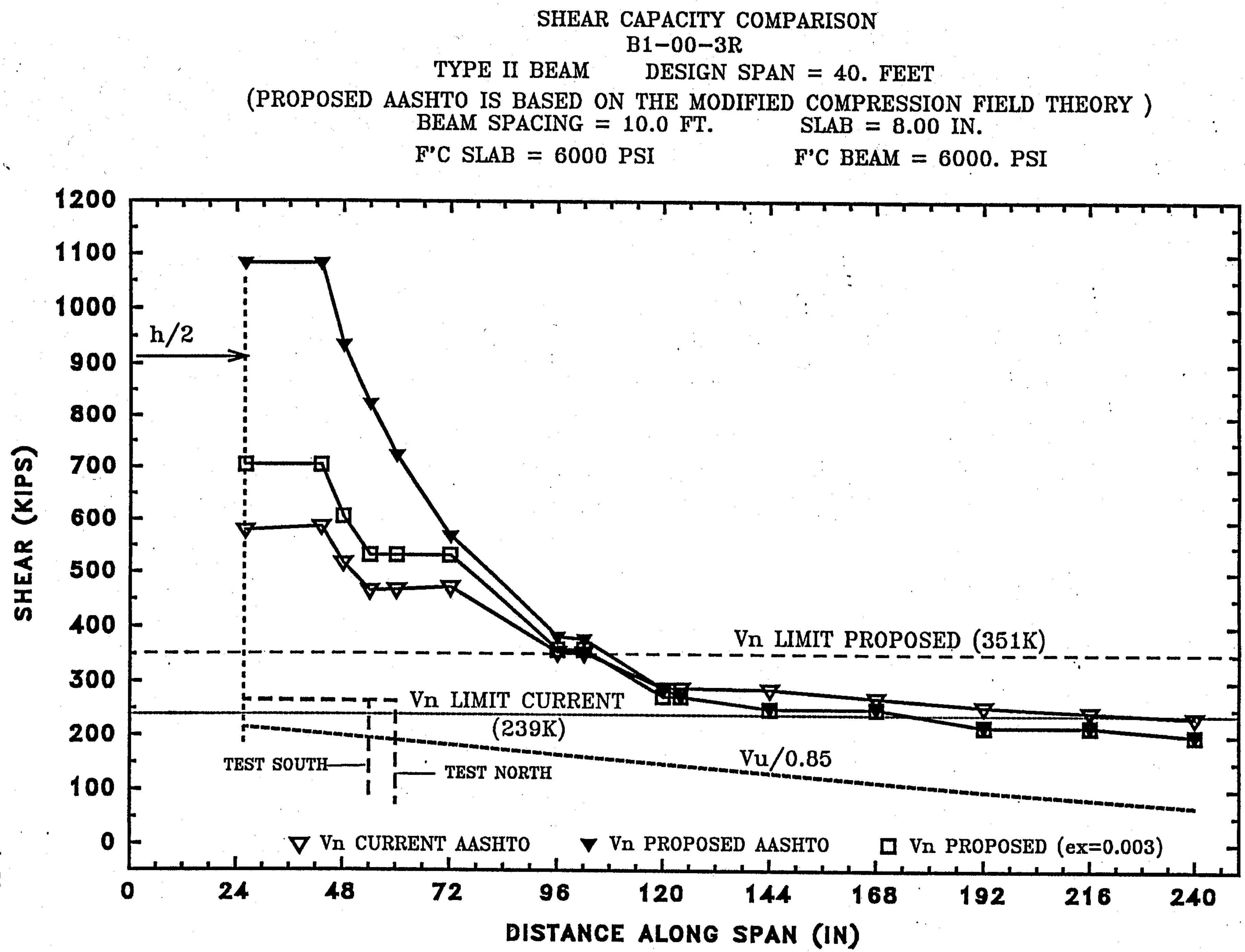
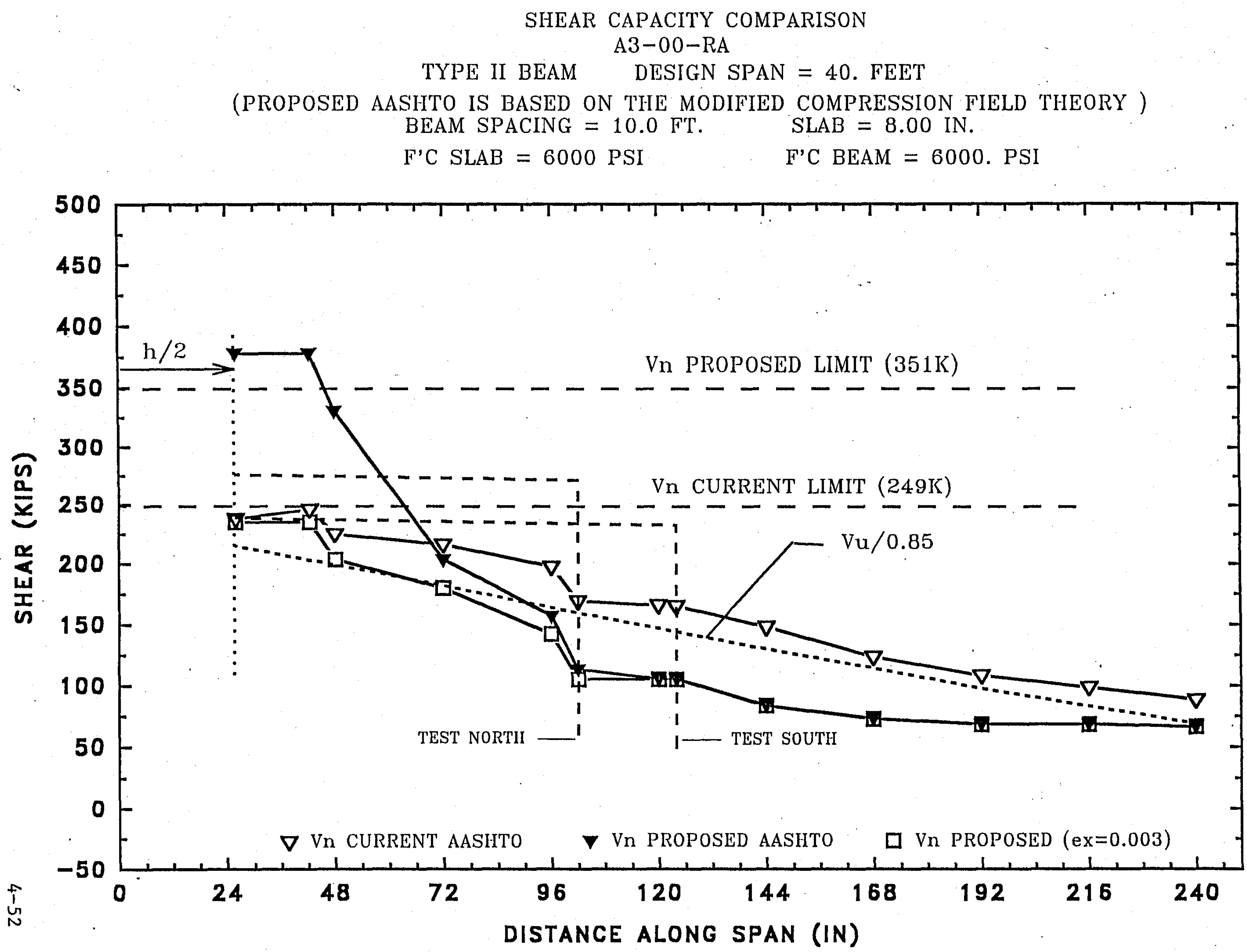
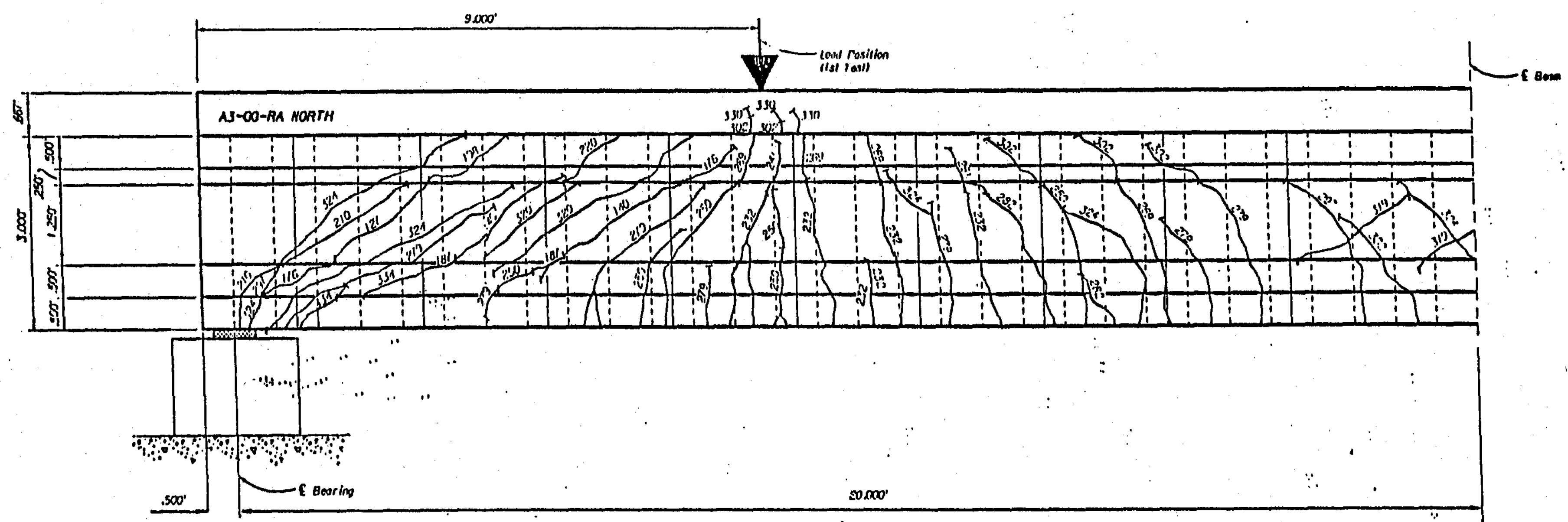

SHEAR CAPACITY COMPARISON
B1-00-3R
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI
SHEAR (KIPS)
1200
1100
1000
900
800
700
600
500
400
300
200
100
0
h/2
Vn LIMIT PROPOSED (351K)
Vn LIMIT CURRENT
(239K)
Vu/0.85
TEST SOUTH
TEST NORTH
Vn CURRENT AASHTO
Vn PROPOSED AASHTO
Vn PROPOSED (ex=0.003)
0
24
48
72
96
120
144
168
192
216
240
DISTANCE ALONG SPAN (IN)

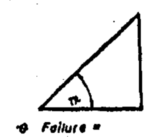
FIGURE 4-33



SHEAR CAPACITY COMPARISON
A3-00-RA
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI
SHEAR (KIPS)
500
450
400
350
300
250
200
150
100
50
0
-50
h/2
Vn PROPOSED LIMIT (351K)
Vn CURRENT LIMIT (249K)
Vu/0.85
TEST NORTH
TEST SOUTH
Vn CURRENT AASHTO
Vn PROPOSED AASHTO
Vn PROPOSED (ex=0.003)
0
24
48
72
96
120
144
168
192
216
240
DISTANCE ALONG SPAN (IN)

FIGURE 4-34





BEAM CRACK CHART
(SPAN 40.00')
A3-00-RA NORTH

NOTE: Heavy Lines indicate Cracks at Failure of Beam.

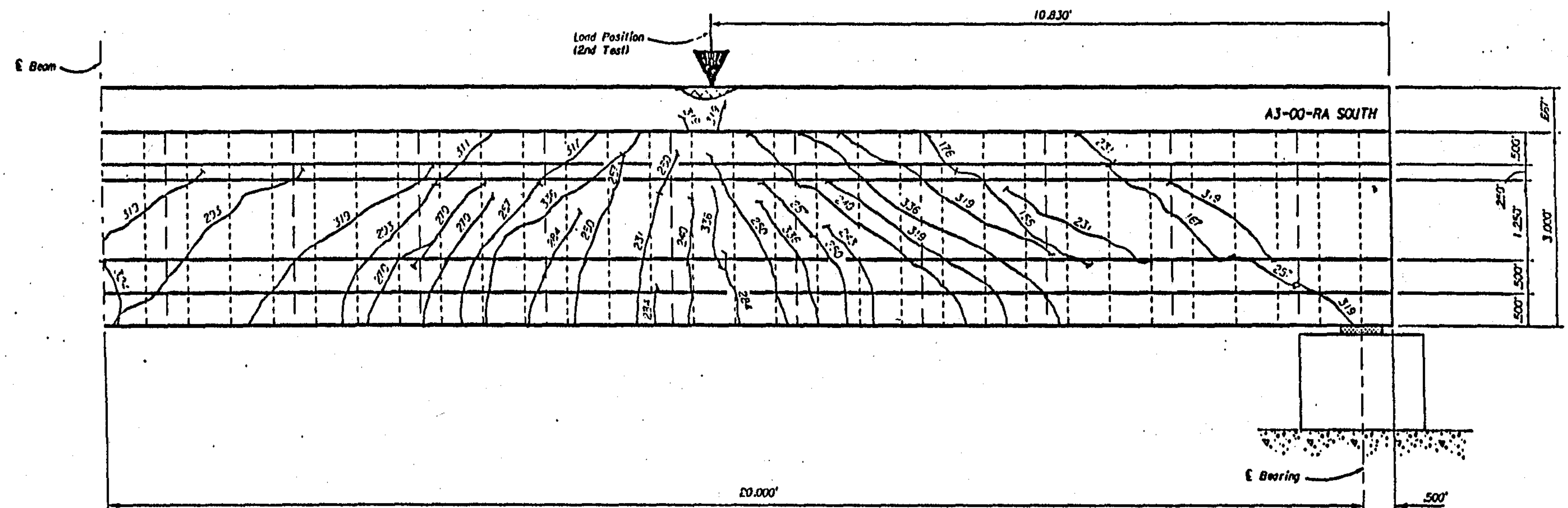


FIGURE 4-35

BEAM CRACK CHART
(SPAN 31.50')
A3-00-RA SOUTH

SHEAR CAPACITY COMPARISON
A3-00-RB
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

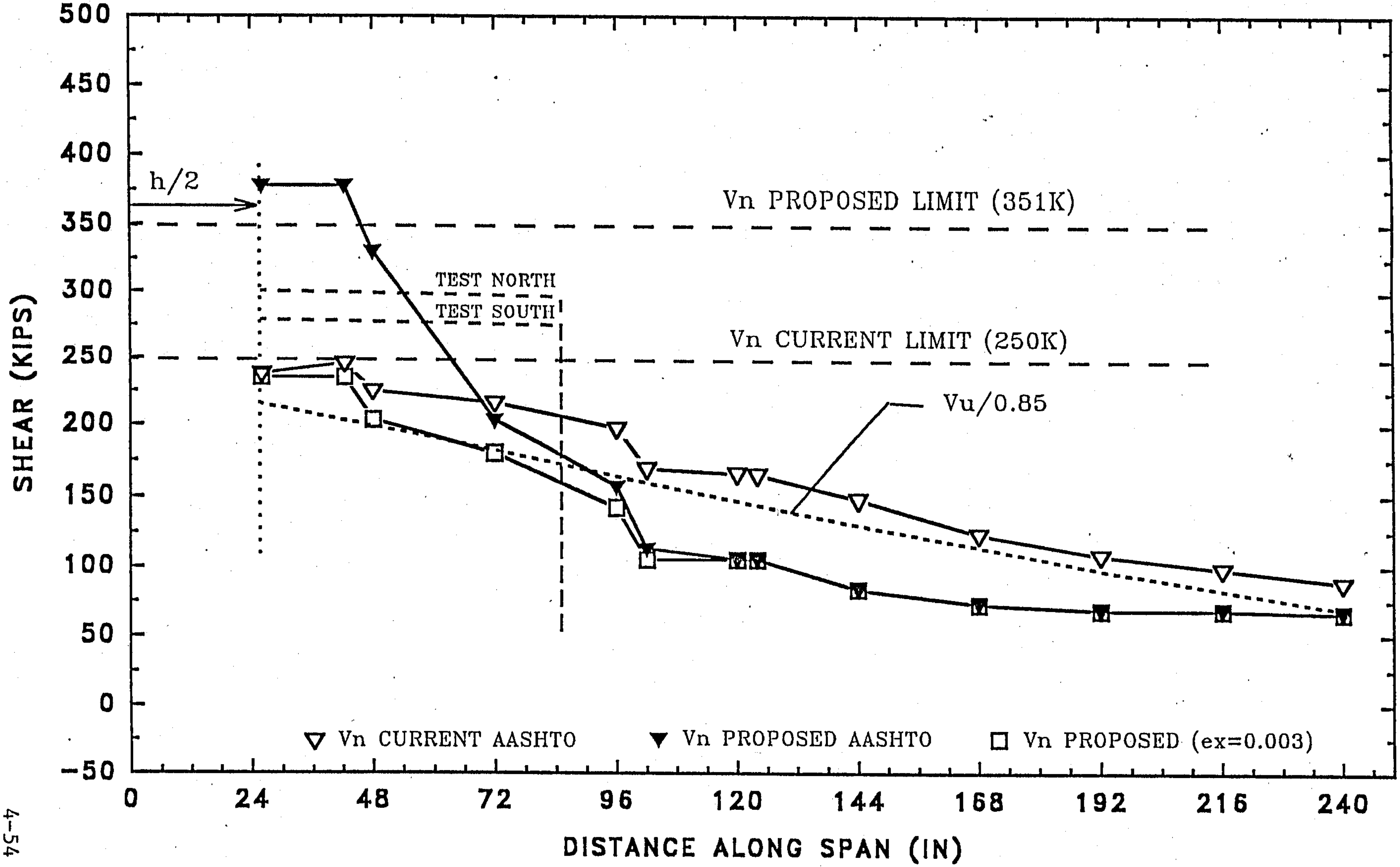
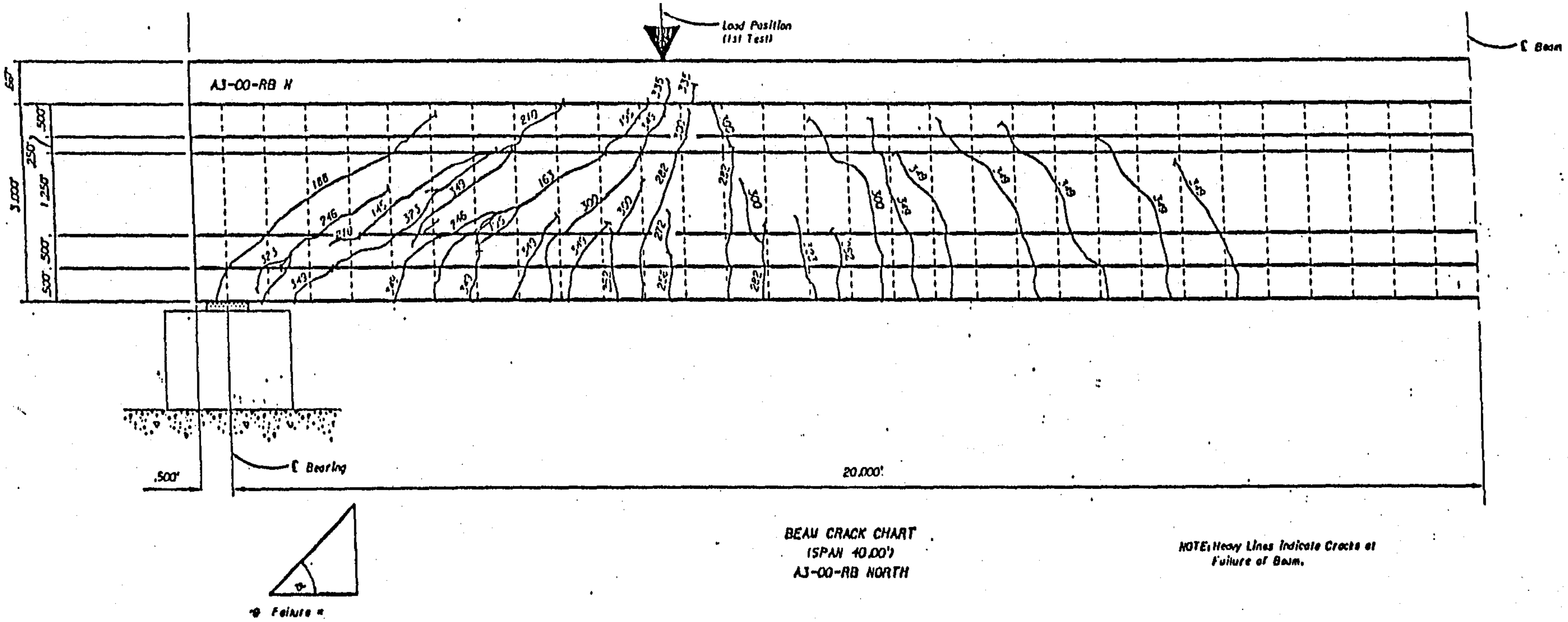
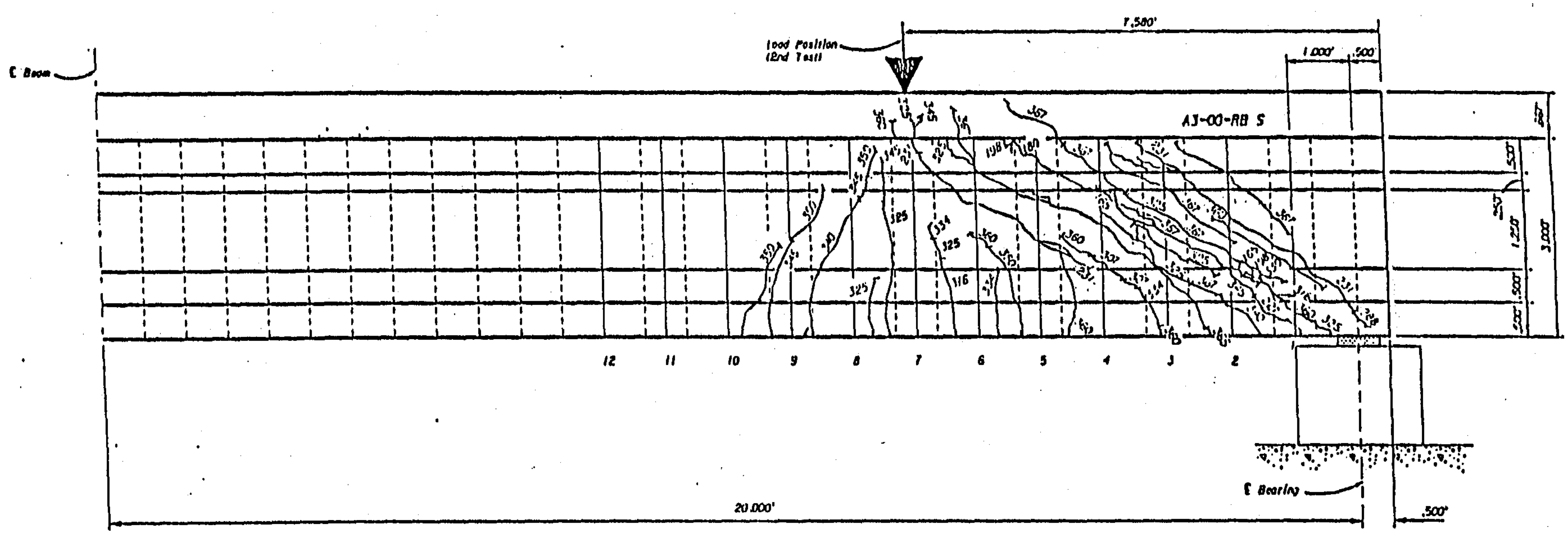


FIGURE 4-36



BEAM CRACK CHART
(SPAN 40.00')
AJ-00-RB NORTH



BEAM CRACK CHART
(SPAN 40.00')
AJ-00-RB SOUTH

FIGURE 4-37

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

The main conclusions from the research are as follows:

1. The current AASHTO code predicts shear capacities for prestressed, concrete girder better than the provisions of the proposed code, although the latter gives a more rational design. approach for shear. Some refinement is required for the proposed method of the code to be acceptable.
2. The provision of confinement steel for the prestressing strands at the end regions of a girder increases its shear capacity.
3. The current AASHTO code predicts shear capacities which are adequate for girders with or without confinement steel .
4. The AASHTO code limits the amount of shear strength which may be provided by shear reinforcement. The limit compared well with the test results.
5. Using a wire mesh for shear reinforcement may provide a significant increase in shear capacity and ductility.
6. Shielding of prestressing strands reduces shear capacity in the end regions of a girder. Girders with 50% of the strands shielded and loaded at 140 inches from the ends exhibited a shear mode of failure with decreased ductility when compared to girders with 25% shielding. Therefore limiting the percentage of shielded strands to 25% appears to be reasonable.

7. The reduction of strength due to the absence of confinement steel was more significant than the reduction in strength due to shielding of prestressing strands.

8. Both the current AASHTO code and the proposed code greatly under-estimate the shear. strength provided by concrete. The current AASHTO code is the less conservative of the two. Since this conclusion was made based on the test results for one girder (B1-00-OR), additional tests are needed to determine better approximations for shear strength provided by the concrete.

5.2 RECOMMENDATIONS

1. Refinement of the proposed AASHTO Code method for shear design is required, since this extensive study shows that it is too conservative.

2. Since the current AASHTO Code provisions for shear design have been shown to be superior in predicting shear strengths, there is no justification in abandoning it for the more conservative proposed provisions based on the Modified Compression Field Theory. The rational approach of the latter method is attractive but requires extensive refinement before adaptation.

3. The study has demonstrated the beneficial effect of confinement steel in delaying bond failure of prestressing strands, and in enhancing shear capacity. It is recommended that provisions of such reinforcement in the end zones of prestressed girders be considered.

REFERENCES

1. American Association of State Highway and Transportation Officials (AASHTO) Executive Committee. Standard Specifications for Highway Bridges, 14th Edition, 1989, Washington, D.C.
2. American Association of State Highway and Transportation Officials (AASHTO) Executive Committee. Interim Specifications - Bridges 1990, Washington, D.C.
3. American Association of State Highway and Transportation Officials (AASHTO) Executive Committee. Interim Specifications - Bridges 1991, Washington, D.C
4. Proposed Revisions to AASHTO Code, June 4, 1991.
5. Collins, Michael P. and Mitchell, Denis (1991). Prestressed Concrete Structures, Prentice-Hall, Englewood Cliffs, New Jersey, 1991.
6. Florida Department of Transportation. Structures Design Guidelines, Bureau of Structures Design, Tallahassee, Florida, 1987, 1991.
7. Florida Department of Transportation. The Design and Analysis of Simple Span Prestressed Beams, "Prestressed Concrete Bridge Girder Design Program -Version 2.3", PSTDSN56, July 1986 (and updates).

Appendix A1

FDOT Prestressed Concrete Bridge Girder Program

Output for Girder A1-00-R

FLORIDA DEPARTMENT OF TRANSPORTATION

PRESTRESSED CONCRETE BRIDGE GIRDER DESIGN PROGRAM - VERSION 2.3

*TEST BEAM REVIEW: A1-00-R TYPE II BEAM SPAN= 40.0 FT
*B. ROBINSON 488-6179
*

**** REVIEW ONLY ***** REVIEW ONLY ***** REVIEW ONLY ***** REVIEW ONLY ***** REVIEW ONLY ***** REVIEW ONLY *****

SSSSSSSSSSSSSSSSSSSSSSSSSSSSSSSSSSSSSSS S T R A I G H T   S T R A N D S SSSSSSSSSSSSSSSSSSSSSSSSSSSSSSSSSSSSSSS

****** L O W - R E L A X A T I O N ******

*** INPUT DATA ***

BEAM TYPE	= 2	UNIT WT. BEAM CONC.	= 150. PCF	STRAND SIZE	= 1/2 IN
SPAN LENGTH	= 40.00 FT	UNIT WT. SLAB CONC.	= 150. PCF	STRAND ULT STRENGTH	= 270 K
BEAM SPACING	= 10.00 FT	28-DAY ST.(SLAB CONC.)	= 6000. PSI	NO. OF WEB STRANDS	= 1
SLAB THICKNESS	= 8.00 IN	E(BM.CONC.)	= 4.00 E(06)PSI	GRID SIZE	= 2.00 IN
L.L. DIST. FACTOR	= 0.91	E(SLAB CONC.)	= 4.00 E(06)PSI	STRAND CL. BOTT. BEAM	= 2.00 IN
TRANS. SLAB WIDTH	= 42.00 IN	E(PRESTRESS STEEL)	= 28.00 E(06)PSI	STRAND CL. SIDE BEAM	= 3.00 IN
UNIF. D.L. N-COMP	= 0.180 KLF	AASHTO L.L. + DIAF. WT.	= HS-20 (+5%)	MAX COMP BM. CONC(ALLOW)	= 2400. PSI
UNIF. D.L. COMP	= 0.359 KLF	RAILROAD L.L.	= E- 0.	MAX. TENSION AT DES. LOAD	= -232. PSI
DIAPHRAGM	= 5% L.L.			COPE DIMENSION	= 0.00 IN

*** SECTION PROPERTIES ***

		PRECAST	COMPOSITE	
AREA	=	369.00	705.00	IN2
WEIGHT	=	384.37	1384.37	PLF
MOMENT OF INERTIA	=	50979.00	155508.70	IN4
YB BEAM	=	15.83	27.35	IN
SECTION MODULUS BOTTOM	=	3220.40	5686.02	IN3
YT BEAM	=	20.17	8.65	IN
SECTION MODULUS TOP	=	2527.47	17976.47	IN3
YTS SLAB	=		16.65	IN
SECTION MODULUS SLAB	=		9339.48	IN3
HEIGHT	=	36.00	44.00	IN

*** BEAM DIMENSIONS (INCHES) ***

B= 18.00 W= 6.00 C= 6.00 E= 6.00 A= 12.00 H= 6.00 G= 3.00 Q= 0.00 R= 0.00

*** CONCENTRATED LOADS APPLIED TO NON-COMPOSITE SECTION ***

LOAD (KIPS)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
DIST. FROM LT. REACT. (FT)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

*** CONCENTRATED STATIC LOADS APPLIED TO COMPOSITE SECTION ***

LOAD (KIPS)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
DIST. FROM LT. REACT. (FT)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

*** CONCENTRATED LIVE LOADS APPLIED TO COMPOSITE SECTION ***

LOAD (KIPS)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
DIST. FROM LT. LOAD (FT)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
LOAD (KIPS)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
DIST. FROM LT. LOAD (FT)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

FLORIDA DEPARTMENT OF TRANSPORTATION

PRESTRESSED CONCRETE BRIDGE GIRDER DESIGN PROGRAM - VERSION 2.3

*TEST BEAM REVIEW: A1-00-R TYPE II BEAM SPAN= 40.0 FT
*B. ROBINSON 488-6179
*

**** REVIEW ONLY ***** REVIEW ONLY ***** REVIEW ONLY ***** REVIEW ONLY ***** REVIEW ONLY ***** REVIEW ONLY *****

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****** L O W - R E L A X A T I O N ******

*** BEAM DESIGN ***

TYPE OF BEAM = 2
NO. OF STRANDS = 16.
SIZE OF STRANDS & PULL = 1/2 IN AT 30990. LB
TYPE OF STRANDS = 270K
ECCENTRICITY AT C.L. = 12.08 IN
ECCENTRICITY AT SUP. = 12.08 IN

CONCRETE RELEASE STRENGTH = 4000. PSI
CONCRETE 28-DAY STRENGTH = 6000. PSI

BEAM END TO CTR BEARING = 6.00 IN

MAX. END TENSION AT REL. = -759. PSI
MAX. CTR TENSION AT REL. = -379. PSI

D.L. DEFLECTION AT MID-SPAN = 0.282 IN (SLAB) 0.000 IN (DIAF)
D.L. DEFLECTION AT 1/4 PT. = 0.201 IN (SLAB) 0.000 IN (DIAF)

ULTIMATE MOMENT REQUIRED = 1696. FT-KIPS
ULTIMATE MOMENT PROVIDED = 2064. FT-KIPS UNDER REINF. RECT. SECT.
1.2 TIMES CRACKING MOMENT = 1427. FT-KIPS

STIRRUP SPACING AT 22.00 INCHES
FROM FACE OF SUPPORT = NO. 4 (GR. 60) AT 3.86 IN. (ONE BAR)

TOP FIBER DESIGN STRESS (C.L.) = 1902. PSI
BOTTOM FIBER DESIGN STRESS (C.L.) = 2482. PSI

PRESTRESS LOSS = 24.02 PERCENT
LOSS AT RELEASE = 10.58 PERCENT

AGE OF BEAM CONCRETE	RELEASE	30 DAYS	60 DAYS	120 DAYS	240 DAYS
NET CAMBER (PSTRES-BEAM) IN.	0.89	1.36	1.46	1.57	1.68
BUILD-UP REQD (CAMBER-SL-DF)IN.	----	1.07	1.18	1.29	1.40
ELASTIC AND TIME-DEPENDENT SHORTENING EFFECTS (EST.) IN.	0.19	0.30	0.33	0.35	0.37

L.L. STRESS IN TOP FIBER OF SLAB AT MIDSPAN = 709. PSI

*** STRAND TRANSFER AND DEVELOPMENT LENGTHS ***

TYPE OF STRAND	AASHTO MULT	TRANSFER LENGTH	DEVELOPMENT LENGTH
FULLY BONDED.........	1.60	3.33 FT	10.58 FT
PATRIALLY DEBONDED... (SHIELDED)	1.00	4.17 FT	13.22 FT

FLORIDA DEPARTMENT OF TRANSPORTATION

PRESTRESSED CONCRETE BRIDGE GIRDER DESIGN PROGRAM - VERSION 2.3

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*TEST BEAM REVIEW: A1-00-R  TYPE II BEAM   SPAN= 40.0 FT
*B. ROBINSON 488-6179
*

***  REVIEW ONLY  *****  REVIEW ONLY  *****  REVIEW ONLY  *****  REVIEW ONLY  *****  REVIEW ONLY  *****  REVIEW ONLY  *****

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                    ****** L O W  -  R E L A X A T I O N ******

                           *** STRAND PATTERN ***

ROW  4 HAS  1. STRAND(S)                   0

ROW  3 HAS  3. STRAND(S)                 0 0 0

ROW  2 HAS  5. STRAND(S)               0 0 0 0 0

ROW  1 HAS  7. STRAND(S)             0 0 0 0 0 0 0

                                        0 : BONDED STRANDS
                                        # : DEBONDED STRAND(S)
```

FLORIDA DEPARTMENT OF TRANSPORTATION

PRESTRESSED CONCRETE BRIDGE GIRDER DESIGN PROGRAM - VERSION 2.3

*TEST BEAM REVIEW: A1-00-R TYPE II BEAM SPAN= 40.0 FT
*B. ROBINSON 488-6179
*

**** REVIEW ONLY ***** REVIEW ONLY ***** REVIEW ONLY ***** REVIEW ONLY ***** REVIEW ONLY ***** REVIEW ONLY *****

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****** L O W - R E L A X A T I O N ******

***** SUMMARY OF RESULTS OF SPECIFICATION COMPARISON *****

A SURVEY OF THE STRAIGHT STRAND ANALYSIS HAS BEEN CONDUCTED TO COMPARE THE REPORTED RESULTS WITH THE DESIGNER'S INPUT OR PROGRAM-DEFAULT SPECIFICATIONS. THE DESIGNER IS CAUTIONED TO VERIFY THAT THE INPUT SPECIFICATIONS ARE PROPER AND IN ACCORDANCE WITH AASHTO AND-OR FDOT ESTABLISHED DESIGN POLICIES PROCEDURES AND REQUIREMENTS. FINAL RESULTS OF THE ANALYSIS ARE REPORTED FOR THE DESIGNER'S REVIEW REGARDLESS OF THE OUTCOME OF THE SPECIFICATION COMPARISON. THE FOLLOWING TABLE OUTLINES THE RESULTS OF THE SPECIFICATION COMPARISON TO ASSIST THE DESIGNER IN HIS REVIEW OF THE ANALYSIS-

DESCRIPTION OF SPECIFICATION COMPARISON	COMPLIES (YES-NO)	SPECIFIED VALUE	ANALYSIS VALUE
1. CRACKING MOMENT (AASHTO ART. 9.18.2)......................	YES	1426.5	2063.6
2. ULTIMATE MOMENT (AASHTO ART. 9.17.1 AND 9.18.1)...........	YES	1696.0	2063.6
3. MAXIMUM NUMBER OF SHIELDED (DEBONDED) STRANDS.............	YES	4	0
4. MINIMUM SPACING OF STRANDS (AASHTO ART. 9.25.2.1)..........	YES	2.000	2.000
5. MAXIMUM TENSILE STRESS (TOP) AT RELEASE (END 15% OF SPAN)..	*NO*	-758.9	-785.9
6. MAXIMUM TENSILE STRESS (TOP) AT RELEASE (MID 70% OF SPAN)..	*NO*	-379.5	-731.1
7. MAXIMUM COMPRESSIVE STRESS (BOTTOM) AT RELEASE.............	*NO*	2400.0	2760.8
8. MAXIMUM COMPRESSIVE STRESS (TOP) AT SERVICE LOAD...........	YES	2400.0	1122.4
9. MAXIMUM TENSILE STRESS (BOTTOM) AT SERVICE LOAD...........	YES	-232.4	-48.8

FLORIDA DEPARTMENT OF TRANSPORTATION

PRESTRESSED CONCRETE BRIDGE GIRDER DESIGN PROGRAM - VERSION 2.3

*TEST BEAM REVIEW: A1-00-R TYPE II BEAM SPAN= 40.0 FT
*B. ROBINSON 488-6179
*

**** REVIEW ONLY ***** REVIEW ONLY ***** REVIEW ONLY ***** REVIEW ONLY ***** REVIEW ONLY ***** REVIEW ONLY *****

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****** L O W - R E L A X A T I O N ******

*** MOMENT SUMMARY (FT-KIPS) *** / *** SHEAR SUMMARY (KIPS) ***

SECTION	DEAD LD.	NCOMP DL	COMP DL	L.L.+I	TOTAL	DEAD LD.	NCOMP DL	COMP DL	L.L.+I	TOTAL
0	0.000	0.000	0.000	0.000	0.000	27.687	3.600	7.180	68.506	106.973
1	99.675	12.960	25.848	238.255	376.738	22.150	2.880	5.744	59.571	90.345
2	177.200	23.040	45.952	405.033	651.225	16.612	2.160	4.308	50.637	73.717
3	232.575	30.240	60.312	500.335	823.462	11.075	1.440	2.872	41.702	57.089
4	265.800	34.560	68.928	551.956	921.244	5.537	0.720	1.436	33.753	41.446
5	276.875	36.000	71.800	551.956	936.631	0.000	0.000	0.000	26.804	26.804
6	265.800	34.560	68.928	551.956	921.244	5.537	0.720	1.436	33.753	41.446
7	232.575	30.240	60.312	500.335	823.462	11.075	1.440	2.872	41.702	57.089
8	177.200	23.040	45.952	405.033	651.225	16.612	2.160	4.308	50.637	73.717
9	99.675	12.960	25.848	238.255	376.738	22.150	2.880	5.744	59.571	90.345
10	0.000	0.000	0.000	0.000	0.000	27.687	3.600	7.180	68.506	106.973
3/4 DEPTH	58.793	7.644	15.246	142.830	224.513	24.573	3.195	6.372	63.480	97.620

**** STRESSES IN EXTREME FIBERS DUE TO EXTERNAL LOADS (LBS PER SQ. IN.) ****

SECTION	BEAM		SLAB		NON-COMP SEC.		TOTAL D.L. NON-COMP SEC.		COMP SECT.		L.L. + IMPACT COMP SEC.		TOTAL	
	TOP	BOT	TOP	BOT	TOP	BOT	TOP	BOT	TOP	BOT	TOP	BOT	TOP	BOT
0	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
1	131.	-103.	342.	-268.	62.	-48.	535.	-420.	17.	-55.	159.	-503.	711.	-977.
2	234.	-183.	608.	-477.	109.	-86.	951.	-746.	31.	-97.	270.	-855.	1252.	-1698.
3	307.	-241.	798.	-626.	144.	-113.	1248.	-979.	40.	-127.	334.	-1056.	1622.	-2163.
4	350.	-275.	912.	-715.	164.	-129.	1426.	-1119.	46.	-145.	368.	-1165.	1841.	-2430.
5	365.	-286.	950.	-745.	171.	-134.	1485.	-1166.	48.	-152.	368.	-1165.	1902.	-2482.
6	350.	-275.	912.	-715.	164.	-129.	1426.	-1119.	46.	-145.	368.	-1165.	1841.	-2430.
7	307.	-241.	798.	-626.	144.	-113.	1248.	-979.	40.	-127.	334.	-1056.	1622.	-2163.
8	234.	-183.	608.	-477.	109.	-86.	951.	-746.	31.	-97.	270.	-855.	1252.	-1698.
9	131.	-103.	342.	-268.	62.	-48.	535.	-420.	17.	-55.	159.	-503.	711.	-977.
10	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
3/4 DEPTH	78.	-61.	202.	-158.	36.	-28.	315.	-248.	10.	-32.	95.	-301.	421.	-581.

FLORIDA DEPARTMENT OF TRANSPORTATION

PRESTRESSED CONCRETE BRIDGE GIRDER DESIGN PROGRAM - VERSION 2.3

*TEST BEAM REVIEW: A1-00-R TYPE II BEAM SPAN= 40.0 FT
*B. ROBINSON 488-6179
*

***** REVIEW ONLY ***** REVIEW ONLY ***** REVIEW ONLY ***** REVIEW ONLY ***** REVIEW ONLY ***** REVIEW ONLY *****

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****** L O W - R E L A X A T I O N ******

**** STRESSES DUE TO EXTERNAL LOADS PLUS PRESTRESS (LBS PER SQ. IN.) ****

	INITIAL PREST.		BEAM PLUS INITIAL PREST. - LOSSES @ REL.		FINAL PREST. PLUS TOT. D.L.(N/C SEC.)		ALL LOADS PLUS FINAL PREST.	
	TOP	BOT	TOP	BOT	TOP	BOT	TOP	BOT
0	-154.	480.	-138.	430.	-18.	55.	-18.	55.
1	-1026.	3203.	-786.	2761.	-245.	2014.	-68.	1456.
2	-1026.	3203.	-684.	2681.	171.	1687.	472.	736.
3	-1026.	3203.	-611.	2623.	468.	1454.	843.	271.
4	-1026.	3203.	-567.	2589.	647.	1314.	1061.	4.
5	-1026.	3203.	-552.	2577.	706.	1268.	1122.	-49.
6	-1026.	3203.	-567.	2589.	647.	1314.	1061.	4.
7	-1026.	3203.	-611.	2623.	468.	1454.	843.	271.
8	-1026.	3203.	-684.	2681.	171.	1687.	472.	736.
9	-1026.	3203.	-786.	2761.	-245.	2014.	-68.	1456.
10	-154.	480.	-138.	430.	-18.	55.	-18.	55.
3/4 DEPTH	-846.	2642.	-679.	2302.	-328.	1760.	-222.	1426.

**** SHEAR VALUES AND STIRRUP SPACING (ONE BAR) ****
(AASHTO 1983 SPECS.)

SECTION	VCI	VCW	VS	SIZE GRADE SPACING
0	0.000	65.393	155.095	NO. 4 (GR. 60) AT 3.86 IN
1	326.795	87.065	100.800	NO. 4 (GR. 60) AT 4.79 IN
2	153.681	104.276	50.968	NO. 4 (GR. 60) AT 9.48 IN
3	94.879	116.568	27.742	NO. 4 (GR. 60) AT 12.00 IN
4	63.604	123.944	28.766	NO. 4 (GR. 60) AT 12.00 IN
5	44.398	126.403	20.130	NO. 4 (GR. 60) AT 12.00 IN
6	63.604	123.944	28.766	NO. 4 (GR. 60) AT 12.00 IN
7	94.879	116.568	27.742	NO. 4 (GR. 60) AT 12.00 IN
8	153.681	104.276	50.968	NO. 4 (GR. 60) AT 9.48 IN
9	326.795	87.065	100.800	NO. 4 (GR. 60) AT 4.79 IN
10	0.000	65.393	155.095	NO. 4 (GR. 60) AT 3.86 IN

**** MAX. ULT. HORIZ. SHEAR (VQ/I) ****
(BETWEEN SL. AND GIR. FLANGE)

SECTION	
0	359.4 PSI
1	311.0 PSI
2	262.7 PSI
3	214.3 PSI
4	170.8 PSI
5	132.3 PSI
6	170.8 PSI
7	214.3 PSI
8	262.7 PSI
9	311.0 PSI
10	359.4 PSI

Appendix A2

Lotus Spreadsheet Results for Girder Al-00-R

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Prestressed Beams Shear Calculations
BEAM-A1-00-R              Type II Beam
FDOT Prestressed Beam Input Data
     Noncomposite Dead Load
      SIP Forms   (psf)=     20.0
      Beam Spacing(ft)=      10.0
        Wnc (klf)      =    180.0
     Composite Dead Load
      Number of Beams =       4.0
      Barrier (plf/ea.)     418.0
      Future Surf(psf)=      15.0
      Curb to Curb(ft.)      40.0
       Wc (klf)    =        359.0

  Concrete Properties
  f'c  (psi) =             6000.0
  Density (pcf) =           145.0
  Ec (x10E6 psi) =           4.02
  B1=     0.75

     PRESTRESSING STEEL
         Ab =    0.153 db =
     f's(ksi)=  270.00     0.500
     fsu(ksi)=  263.43
     fse(ksi)=  154.00
LOW RELAX.      GAMMA= 0.28
STRESS RELIEV. GAMMA= 0.40
         ENTER GAMMA=        0.28
ROW    Area        n          y
       (in2)                (in)
   1    1.071         7      2.00
   2    0.765         5      4.00
   3    0.459         3      6.00
   4    0.153         1      8.00
   5
   6
     -------- -------- ----------
           n=        16

 As=     2.448      yna=     3.75
                      d=    40.25
 a = 3.010645      rho= 0.001448
                     dv=    38.74

TRANSFER LENGTH (IN)
 lt = 50db     (IN)=        25.00
 END DISTANCE (IN)=          6.00
DEBONDED STRANDS (UNITS = INCHES)
 ndb=         0       nb=        16
 Adb=         0       Ab=     2.448
 ldb=      0.00   (FROM END OF BEAM)
dvdb=    38.74
```

BEAM -- A1-00-R

Prestressed Beam Program Results

Type II Beam Loaded with Hs20 truck + 5%

Span length, L (ft) = 40.00 Flange, b (in) 42.00

Shear Depth, dv (in) = 38.74 Web, b' (in) 6.00

Depth , h (in) = 44.00

SHEAR

	x/L	Location (in)		V DEAD (K)	V LIVE (K)	Vu (K)	Vu/0.9 (K)	Vu/0.85 (K)
(0.00 L)	0.000	0.00	0	38.5	68.5	198.5	220.5	233.5
(0.10 L)	0.100	48.00	1	30.8	59.6	169.2	188.0	199.0
(0.20 L)	0.200	96.00	2	23.1	50.6	139.7	155.2	164.3
(0.30 L)	0.300	144.00	3	15.4	41.7	110.4	122.6	129.8
(0.40 L)	0.400	192.00	4	7.7	33.8	83.2	92.5	97.9
(0.50 L)	0.500	240.00	5	0.0	26.8	58.1	64.5	68.3
(h/2+ 4)	0.054	26.00		34.3	63.7	182.6	202.9	214.8
(dv + 4)	0.089	42.74		31.6	60.6	172.4	191.5	202.8
(0.15 L)	0.150	72.00		27.0	55.1	154.4	171.6	181.7
(0.25 L)	0.250	120.00		19.3	46.2	125.0	138.9	147.1
(0.35 L)	0.350	168.00		11.6	37.8	96.8	107.6	113.9
(0.45 L)	0.450	216.00		3.8	30.3	70.7	78.5	83.1
LD.PT.N	0.213	102.00		22.1	49.5	136.0	151.1	160.0
LD.PT.S	0.258	124.00		18.6	45.4	122.6	136.2	144.2

MOMENT

	x/L	Location (in)		M DEAD (K)	M LIVE (K)	Mu (K)	Mu/0.9 (K)	Mu/0.85 (K)
(0.00 L)	0.000	0.00	0	0.0	0.0	0.0	0.0	0.0
(0.10 L)	0.100	48.00	1	138.5	238.3	696.4	773.7	819.3
(0.20 L)	0.200	96.00	2	246.2	405.0	1197.6	1330.6	1408.9
(0.30 L)	0.300	144.00	3	323.2	500.3	1504.1	1671.3	1769.6
(0.40 L)	0.400	192.00	4	369.3	552.0	1676.1	1862.3	1971.9
(0.50 L)	0.500	240.00	5	384.7	552.0	1696.1	1884.6	1995.4
(h/2+ 4)	0.054	26.00		75.0	129.1	377.2	419.1	443.8
(dv + 4)	0.089	42.74		123.3	212.2	620.1	689.0	729.6
(0.15 L)	0.150	72.00		192.3	321.7	947.0	1052.2	1114.1
(0.25 L)	0.250	120.00		284.7	452.7	1350.9	1500.9	1589.2
(0.35 L)	0.350	168.00		346.3	526.2	1590.1	1766.8	1870.7
(0.45 L)	0.450	216.00		377.0	552.0	1686.1	1873.4	1983.6
LD.PT.N	0.213	102.00		255.8	416.9	1235.9	1373.2	1454.0
LD.PT.S	0.258	124.00		291.1	460.6	1376.4	1529.3	1619.3

SHEAR STRENGTH PROVIDED --CURRENT AASHTO

BEAM -- A1-00-R

STRENGTH PROVIDED BY CONCRETE, Vc					STRENGTH STEEL, Vs			SHEAR CAPACITY
FDOT PRESTRESSED BEAM PROGRAM					fy =60.00 d =40.25			Vn (K)
	x/L	Location (in)		Vc (K)	Av	s	Vs	Vn
(0.00 L)	0.000	0.00	0	65.0	0.2	3	161.0	226.0
(0.10 L)	0.100	48.00	1	87.0	0.4	7	138.0	225.0
(0.20 L)	0.200	96.00	2	104.0	0.3	8	90.6	194.6
(0.30 L)	0.300	144.00	3	95.0	0.2	10	48.3	143.3
(0.40 L)	0.400	192.00	4	64.0	0.2	12	40.3	104.3
(0.50 L)	0.500	240.00	5	44.0	0.2	12	40.3	84.3
(h/2+ 4)	0.054	26.00		76.9	0.4	6	161.0	237.9
(dv + 4)	0.089	42.74		84.6	0.4	6	161.0	245.6
(0.15 L)	0.150	72.00		95.5	0.4	8	120.8	216.3
(0.25 L)	0.250	120.00		99.5	0.2	8	60.4	159.9
(0.35 L)	0.350	168.00		79.5	0.2	12	40.3	119.8
(0.45 L)	0.450	216.00		54.0	0.2	12	40.3	94.3
LD.PT.N	0.213	102.00		102.9	0.3	8	90.6	193.4
LD.PT.S	0.258	124.00		98.8	0.2	8	60.4	159.1

SHEAR CAPACITY MCFT (epsilon x computed)
BEAM -- A1-00-R (ASSUMPTION: EsAs = 0.0)

phi=	fse=	f'c=	bv=	dv=	Aps=	fy=	Ep=
0.85	154	6000	6.0	38.74	2.448	60.0	28000

CAPACITY			DISTANCE	FROM CENTERLINE	OF SUPPORT		
VALUES	(X/L)	0.000	0.100	0.200	0.300	0.400	0.500
	(IN)	0.000	48.000	96.000	144.000	192.000	240.000
Vu, kips		198.467	169.173	139.663	110.370	83.243	58.067
Mu, kip-ft		0.000	696.367	1197.560	1504.143	1676.090	1696.110
Nu, kips		0.000	0.000	0.000	0.000	0.000	0.000
Vp, kips		0.000	0.000	0.000	0.000	0.000	0.000
dv, in		38.74	38.74	38.74	38.74	38.74	38.74
v/f'c		0.167	0.143	0.118	0.093	0.070	0.049
(ex x1000)		1.188	-0.216	1.422	2.691	3.125	2.898
THETA i		30.000	30.000	34.000	30.000	30.000	30.000
THETA		34.000	25.000	34.000	38.000	43.000	46.000
BETA		1.08	2.53	1.30	1.00	1.25	1.39
Av, in2		0.2	0.4	0.3	0.2	0.2	0.2
s, in		3.00	7.00	8.00	10.00	12.00	12.00
Vc, kips		19.4	45.6	23.4	18.0	22.5	25.0
Vs, kips		229.8	284.9	129.2	59.5	41.5	37.4
Vn, kips		249.2	330.4	152.7	77.5	64.1	62.4

SHEAR CAPACITY MCFT (epsilon x computed)
BEAM -- A1-00-R

0.054	0.089	0.150	0.250	0.350	0.450	0.213	0.258
26.000	42.745	72.000	120.000	168.000	216.000	102.000	124.000
182.599	172.381	154.418	125.017	96.807	70.655	136.002	122.576
377.199	620.124	946.963	1350.852	1590.117	1686.100	1235.883	1376.400
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
38.74	38.74	38.74	38.74	38.74	38.74	38.74	38.74
0.154	0.145	0.130	0.105	0.082	0.060	0.115	0.103
-1.489	-0.520	0.730	2.183	2.908	3.011	1.803	2.268
30.000	30.000	30.000	30.000	30.000	30.000	30.000	30.000
27.000	25.000	34.000	35.000	38.000	43.000	34.000	35.000
2.16	2.53	1.73	0.77	1.00	1.25	1.04	0.77
0.4	0.4	0.4	0.2	0.2	0.2	0.3	0.2
6.00	6.00	8.00	8.00	12.00	12.00	8.00	8.00
38.9	45.6	31.2	13.9	18.0	22.5	18.7	13.9
304.2	332.4	172.3	83.0	49.6	41.5	129.2	83.0
343.1	377.9	203.5	96.9	67.6	64.1	148.0	96.9

SHEAR CAPACITY MCFT (epsilon x =0.003)
BEAM --

phi=	f'c=	bv=	dv=	Aps=	fy=	Ep=
0.85	6000	6	38.74467	2.448	60	28000

CAPACITY	DISTANCE FROM	CENTERLINE OF SUPPORT				
VALUES	(X/L)	0.000	0.100	0.200	0.300	0.400	0.500
	(IN)	0.000	48.000	96.000	144.000	192.000	240.000
Vu, kips		198.467	169.173	139.663	110.370	83.243	58.067
Mu, kip-ft		0.000	696.367	1197.560	1504.143	1676.090	1696.110
Nu, kips		0.000	0.000	0.000	0.000	0.000	0.000
Vp, kips		0.000	0.000	0.000	0.000	0.000	0.000
dv, in		38.74	38.74	38.74	38.74	38.74	38.74
v/f'c		0.167	0.143	0.118	0.093	0.070	0.049
(ex x1000)		3.000	3.000	3.000	3.000	3.000	3.000
THETA		41.000	35.000	35.000	38.000	43.000	46.000
BETA		0.990	0.770	0.770	1.000	1.250	1.390
Av, in2		0.2	0.4	0.3	0.2	0.2	0.2
s, in		3.00	7.00	8.00	10.00	12.00	12.00
Vc, kips		17.8	13.9	13.9	18.0	22.5	25.0
Vs, kips		178.3	189.7	124.5	59.5	41.5	37.4
Vn, kips		196.1	203.6	138.4	77.5	64.1	62.4

SHEAR CAPACITY MCFT (epsilon x =0.003)
BEAM -- A1-00-R

0.054	0.089	0.150	0.250	0.350	0.450	0.213	0.258
26.000	42.745	72.000	120.000	168.000	216.000	102.000	124.000
182.599	172.381	154.418	125.017	96.807	70.655	136.002	122.576
377.199	620.124	946.963	1350.852	1590.117	1686.100	1235.883	1376.400
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
38.74	38.74	38.74	38.74	38.74	38.74	38.74	38.74
0.154	0.145	0.130	0.105	0.082	0.060	0.115	0.103
3.000	3.000	3.000	3.000	3.000	3.000	3.000	3.000
41.000	35.000	35.000	35.000	38.000	43.000	35.000	35.000
0.990	0.770	0.770	0.770	1.000	1.250	0.770	0.770
0.4	0.4	0.4	0.2	0.2	0.2	0.3	0.2
6.00	6.00	8.00	8.00	12.00	12.00	8.00	8.00
17.8	13.9	13.9	13.9	18.0	22.5	13.9	13.9
178.3	221.3	166.0	83.0	49.6	41.5	124.5	83.0
196.1	235.2	179.9	96.9	67.6	64.1	138.4	96.9

SHEAR CAPACITY -- RESPONSE INPUT
BEAM -- A1-00-R

Concrete Properties

f'c (psi) =	6000
Density (pcf) =	145
Ec (x10E6 psi) =	4.02
e'c =	0.003
fcr (psi)= 4(SQRT(f'c))=	309.8

Prestressing Steel Properties

f's(ksi)=	270.00
fsu(ksi)=	263.43
fse(ksi)=	154.00
Pjack (lbs) =	31000.00
Ep (x10E6 psi) =	28.00
Tension Stiffening Factor =	0.49
Prestrain =	0.0072

n =	16	d (in)=	40.25
Astrand =	0.153	dv (in)=	38.74
As (in2)=	2.448	pho =	0.001448
yna (in)=	3.75	a =	3.01

Dist. to web strain, ex =	23.12
Dist. to center of web =	23.12

RAMBERG OSGOOD FACTORS

LOW RELAXATION STEEL

A =	0.025
B =	118.0
C =	10.0

STRESS RELIEVED STEEL

A =	0.030
B =	121.0
C =	6.0

CRACK SPACING
BEAM --A1-00-R

LONGITUDINAL CRACK SPACING, Smx
Smx = 51.0
Stirrups:(#4 bars, fy = 60 ksi)
dbx = 0.500
sx = 2.00
cx = 12.50
Asx(in2)= 0.306
Apx(in2)= 2.448
Ac (in) =705.00
k1 = 0.80

TRANSVERSE CRACK SPACING, Smv

Smv dbv(in) 0.500
bw(in)= 6.00

k1= 0.4	x/L	Locatio (in)	Av (in2)	s (in)	cv (in)	Smv (in)	THETA	Sm0
(0.00 L)	0.000	0.00	0.2	3.00	3.50	12.1	34	12.6
(0.10 L)	0.100	48.00	0.4	7.00	2.00	10.7	25	10.7
(0.20 L)	0.200	96.00	0.3	8.00	2.75	15.1	34	15.2
(0.30 L)	0.300	144.00	0.2	10.00	3.50	24.0	38	22.3
(0.40 L)	0.400	192.00	0.2	12.00	3.50	27.4	43	25.0
(0.50 L)	0.500	240.00	0.2	12.00	3.50	27.4	46	25.3
(h/2+ 4)	0.054	26.00	0.4	6.00	2.00	9.7	27	9.9
(dv + 4)	0.089	42.74	0.4	6.00	2.00	9.7	25	9.8
(0.15 L)	0.150	72.00	0.4	8.00	2.00	11.6	34	12.1
(0.25 L)	0.250	120.00	0.2	8.00	3.50	20.6	35	19.6
(0.35 L)	0.350	168.00	0.2	12.00	3.50	27.4	38	24.5
(0.45 L)	0.450	216.00	0.2	12.00	3.50	27.4	43	25.0
LD.PT.N	0.213	102.00	0.3	8.00	2.75	15.1	34	15.2
LD.PT.S	0.258	124.00	0.2	8.00	3.50	20.6	35	19.6

TEST LOAD SHEAR --- NORTH END

BEAM -- A1-00-R

TEST SPAN L (ft)	LOAD POINT a (in)	AREA SECTION A (in2)	DEAD LOAD w (klf)	TEST LOAD P (K)	a/d
40.00	102.00	705.00	0.73	255.50	2.53

	x/L	Location (in)	V DEAD (K)	V LIVE (K)	V TOTAL (K)
(LOAD PT)	0.213	102.00	8.45	201.21	209.65
(0.00 L)	0.000	0.00	14.69	201.21	215.89
(0.10 L)	0.100	48.00	11.75	201.21	212.96
(0.20 L)	0.200	96.00	8.81	201.21	210.02
(0.30 L)	0.300	144.00	5.88	-54.29	-48.42
(0.40 L)	0.400	192.00	2.94	-54.29	-51.36
(0.50 L)	0.500	240.00	0.00	-54.29	-54.29
(h/2+ 4)	0.054	26.00	13.10	201.21	214.30
(dv + 4)	0.089	42.74	12.07	201.21	213.28
(0.15 L)	0.150	72.00	10.28	201.21	211.49
(0.25 L)	0.250	120.00	7.34	-54.29	-46.95
(0.35 L)	0.350	168.00	4.41	-54.29	-49.89
(0.45 L)	0.450	216.00	1.47	-54.29	-52.83

TEST LOAD SHEAR --- SOUTH END

BEAM--- A1-00-R

TEST SPAN L (ft)	LOAD POINT a (in)	AREA SECTION A (in2)	DEAD LOAD w (klf)	TEST LOAD P (K)	a/d
31.50	124.00	705.00	0.73	304.00	3.08

	x/L	Location (in)	V DEAD (K)	V LIVE (K)	V TOTAL (K)
(LOAD PT)	0.258	124.00	3.98	204.28	208.25
(0.00 L)	0.000	0.00	11.57	204.28	215.84
(0.10 L)	0.100	48.00	8.63	204.28	212.90
(0.20 L)	0.200	96.00	5.69	204.28	209.97
(0.30 L)	0.300	144.00	2.75	-99.72	-96.97
(0.40 L)	0.400	192.00	-0.18	-99.72	-99.91
(0.50 L)	0.500	240.00	-3.12	-99.72	-102.85
(h/2+ 4)	0.054	26.00	9.98	204.28	214.25
(dv + 4)	0.089	42.74	8.95	204.28	213.23
(0.15 L)	0.150	72.00	7.16	204.28	211.44
(0.25 L)	0.250	120.00	4.22	204.28	208.50
(0.35 L)	0.350	168.00	1.29	-99.72	-98.44
(0.45 L)	0.450	216.00	-1.65	-99.72	-101.38

TEST LOAD MOMENT --- NORTH END

BEAM -- A1-00-R

TEST SPAN L (ft)	LOAD POINT a (in)	AREA SECTION A (in2)	DEAD LOAD w (klf)	TEST LOAD P (K)	a/d
40.00	102.00	705.00	0.73	255.50	2.53

	x/L	Location (in)	M DEAD (K)	M LIVE (K)	M TOTAL (K)
(LOAD PT)	0.213	102.00	98.31	1710.25	1808.57
(0.00 L)	0.000	0.00	0.00	0.00	0.00
(0.10 L)	0.100	48.00	52.88	804.83	857.70
(0.20 L)	0.200	96.00	94.00	1609.65	1703.65
(0.30 L)	0.300	144.00	123.38	1520.23	1643.60
(0.40 L)	0.400	192.00	141.00	1303.05	1444.05
(0.50 L)	0.500	240.00	146.88	1085.88	1232.75
(h/2+ 4)	0.054	26.00	30.10	435.95	466.05
(dv + 4)	0.089	42.74	47.66	716.71	764.37
(0.15 L)	0.150	72.00	74.91	1207.24	1282.14
(0.25 L)	0.250	120.00	110.16	1628.81	1738.97
(0.35 L)	0.350	168.00	133.66	1411.64	1545.29
(0.45 L)	0.450	216.00	145.41	1194.46	1339.87

TEST LOAD MOMENT --- SOUTH END

BEAM--- A1-00-R

TEST SPAN LT (ft)	LOAD POINT a (in)	AREA SECTION A (in2)	DEAD LOAD w (klf)	TEST LOAD P (K)	a/d
31.50	124.00	705.00	0.73	304.00	3.08

	x/L	Location (in)	M DEAD (K)	M LIVE (K)	M TOTAL (K)
(LOAD PT)	0.258	124.00	80.31	2110.84	2191.16
(0.00 L)	0.000	0.00	0.00	0.00	0.00
(0.10 L)	0.100	48.00	40.39	817.10	857.49
(0.20 L)	0.200	96.00	69.03	1634.20	1703.23
(0.30 L)	0.300	144.00	85.92	1944.63	2030.56
(0.40 L)	0.400	192.00	91.06	1545.74	1636.80
(0.50 L)	0.500	240.00	84.45	1146.84	1231.29
(h/2+ 4)	0.054	26.00	23.34	442.60	465.93
(dv + 4)	0.089	42.74	36.54	727.64	764.18
(0.15 L)	0.150	72.00	56.18	1225.65	1281.83
(0.25 L)	0.250	120.00	78.95	2042.75	2121.70
(0.35 L)	0.350	168.00	89.96	1745.19	1835.15
(0.45 L)	0.450	216.00	89.23	1346.29	1435.51

Appendix A3

RESPONSE Program Input and Output for

Girder A1-00-R at 0.3L

```
esponse Version 1 Data-File
opyright 1990 A. Felber
ame of Section:                        A1-00-R
its M/U 'Metric/U.S.Customary':             U
umber of Concrete Types (1-5):             3
 Type          f'c         ec'        fcr     Tension Stiffening
 Number       [psi]  [Milli-Strain] [psi]    Factor
     1       6000    -3.000       310       0.49
     2       6000    -3.000       310       0.00
     3       6000    -3.000       310       0.00
umber of Rebar Types  (1-5):               1
 Type  Elastic Modulus   fy        esh       esrupt       fu
 Number    [ksi]        [ksi]    [---Milli-Strain--]  [ksi]
     1  29000.00     60.00    10.000    40.000     60.00
umber of Tendon Types  (1-5):              1
 Type     [Ramberg-Osgood-Factors--]     Elastic Modulus   fpu   eprupt
 Number        A         B        C           [ksi]         [ksi][Milli-Strain]
     1      0.025   118.000    10.000  28000.00     270.00    54.000
eight of Section:                       44.00      in
istance to Moment Axis:                 27.35      in
hear Y/N 'Yes/No':                          Y
 Web width  (bw)   :                     6.00      in
 Shear depth (jd) :                     38.74      in
 Distance to web strain ex :            23.12      in
 Distance to center of web :            23.12      in
 Longitudinal crack spacing:            51.00      in
 Maximum Aggregate size    :             0.75      in
tirrups Y/N 'Yes/No':                       Y
 Transverse crack spacing  :            24.00      in
 Area of Stirrups (Av)     :             0.20      in^2
 Stirrup Spacing  (s)      :            10.00      in
 Stirrup (Rebar) Type      :              1
umber of Concrete Layers (1-20):          6
 Layer        y      bottom width  top width   height     Type
 Number      [in]        [in]         [in]       [in]      Number
     1       0.00     18.00      18.00      6.00        1
     2       6.00     18.00       6.00      6.00        1
     3      12.00      6.00       6.00     15.00        1
     4      27.00      6.00      12.00      3.00        1
     5      30.00     12.00      12.00      6.00        1
     6      36.00     42.00      42.00      8.00        1
umber of Rebar Layers   (0-10) :          0
umber of Tendon Layers (0-10) :           4
 Layer          y           Area       Prestrain     Type
 Number        [in]        [in^2]   [Milli-Strain] Number
     1        2.00      1.07      7.200        1
     2        4.00      0.77      7.200        1
     3        6.00      0.46      7.200        1
     4        8.00      0.15      7.200        1
onsider displaced Concrete Y/N:             N
hermal & Shrinkage Strains Y/N :            N
nitial Strains             Y/N :            N
```

ESPONSE OUTPUT FILE
NALYSIS: Shear & Constant Axial-Load
ECTION NAME : A1-00-R
ONCRETE MODEL: Parabolic Material Factors: C:0.600 S:0.850 P:0.900
ENSION-STIFF.: No THEORY: MCFT FACTORED: No ACCURACY: well_done
xial-Load: 0.00kip Moment: 0.00kft Shear: 0.00kip
dN/dM: 0.00 dN/dV: 0.00 dM/dV: 0.00

Axial-Load	Shear	Moment	ex	et	e1	e2	theta
kips	kips	ft*kip	[------Milli-Strain------]				degrees
-0.0	18.1	941.6	-0.18	-0.00	0.00	-0.19	7.20
0.0	31.8	941.9	-0.18	-0.00	0.01	-0.19	12.16
0.5	57.6	942.8	-0.18	-0.00	0.03	-0.21	19.64
-0.5	112.0	945.7	-0.19	-0.00	0.08	-0.27	28.83
0.1	77.3	927.1	-0.17	0.17	0.23	-0.23	21.32
0.1	81.1	921.2	-0.16	0.32	0.40	-0.24	21.21
0.1	93.0	906.8	-0.14	0.66	0.80	-0.29	21.36
0.1	106.5	891.8	-0.12	0.99	1.20	-0.33	21.61
0.0	120.7	876.3	-0.11	1.33	1.60	-0.38	21.85
0.4	134.4	862.7	-0.09	1.65	2.00	-0.44	22.17
0.4	144.8	851.6	-0.08	1.98	2.40	-0.50	22.39
0.0	144.1	846.4	-0.07	2.34	2.80	-0.53	21.88
0.0	143.9	842.3	-0.07	2.70	3.20	-0.57	21.44
-0.0	143.9	838.7	-0.06	3.06	3.60	-0.61	21.08
-0.1	143.9	835.5	-0.06	3.42	4.00	-0.64	20.77
-0.3	143.9	832.6	-0.06	3.78	4.40	-0.68	20.51
0.0	144.0	830.1	-0.05	4.14	4.80	-0.72	20.29
0.3	144.1	827.8	-0.05	4.50	5.20	-0.75	20.09
0.0	144.2	825.8	-0.05	4.86	5.60	-0.79	19.92
0.2	144.4	823.9	-0.05	5.22	6.00	-0.83	19.77
-0.0	144.5	822.3	-0.04	5.58	6.40	-0.86	19.64
-0.0	144.5	820.9	-0.04	5.94	6.80	-0.90	19.53
0.0	144.6	819.6	-0.04	6.30	7.20	-0.94	19.42
0.0	144.7	818.4	-0.04	6.66	7.60	-0.98	19.33
-0.0	144.7	817.4	-0.04	7.02	8.00	-1.02	19.25
-0.0	144.8	816.5	-0.04	7.38	8.40	-1.06	19.18
-0.0	144.8	815.7	-0.04	7.74	8.80	-1.10	19.12
0.0	144.8	815.0	-0.04	8.10	9.20	-1.14	19.06
0.3	144.7	814.4	-0.04	8.46	9.60	-1.18	19.01
0.3	144.7	813.9	-0.03	8.81	10.00	-1.22	18.97
-0.0	144.7	813.6	-0.03	9.17	10.40	-1.26	18.93
-0.0	144.6	813.3	-0.03	9.53	10.80	-1.30	18.90
-0.5	144.5	813.1	-0.03	9.89	11.20	-1.35	18.87
0.0	144.4	812.9	-0.03	10.24	11.60	-1.39	18.85
0.3	144.3	812.8	-0.03	10.60	12.00	-1.43	18.83
0.2	144.2	812.7	-0.03	10.96	12.40	-1.48	18.82
-0.0	144.0	812.8	-0.03	11.31	12.80	-1.52	18.81
-0.0	143.8	812.9	-0.03	11.67	13.20	-1.57	18.80

ND OF RESPONSE OUTPUT FILE

Appendix A4

Plots of Shear Capacity Comparison

SHEAR CAPACITY COMPARISON
A0-00-R
TYPE II BEAM   DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT.   SLAB = 8.00 IN.
F'C SLAB = 6000 PSI   F'C BEAM = 6000. PSI

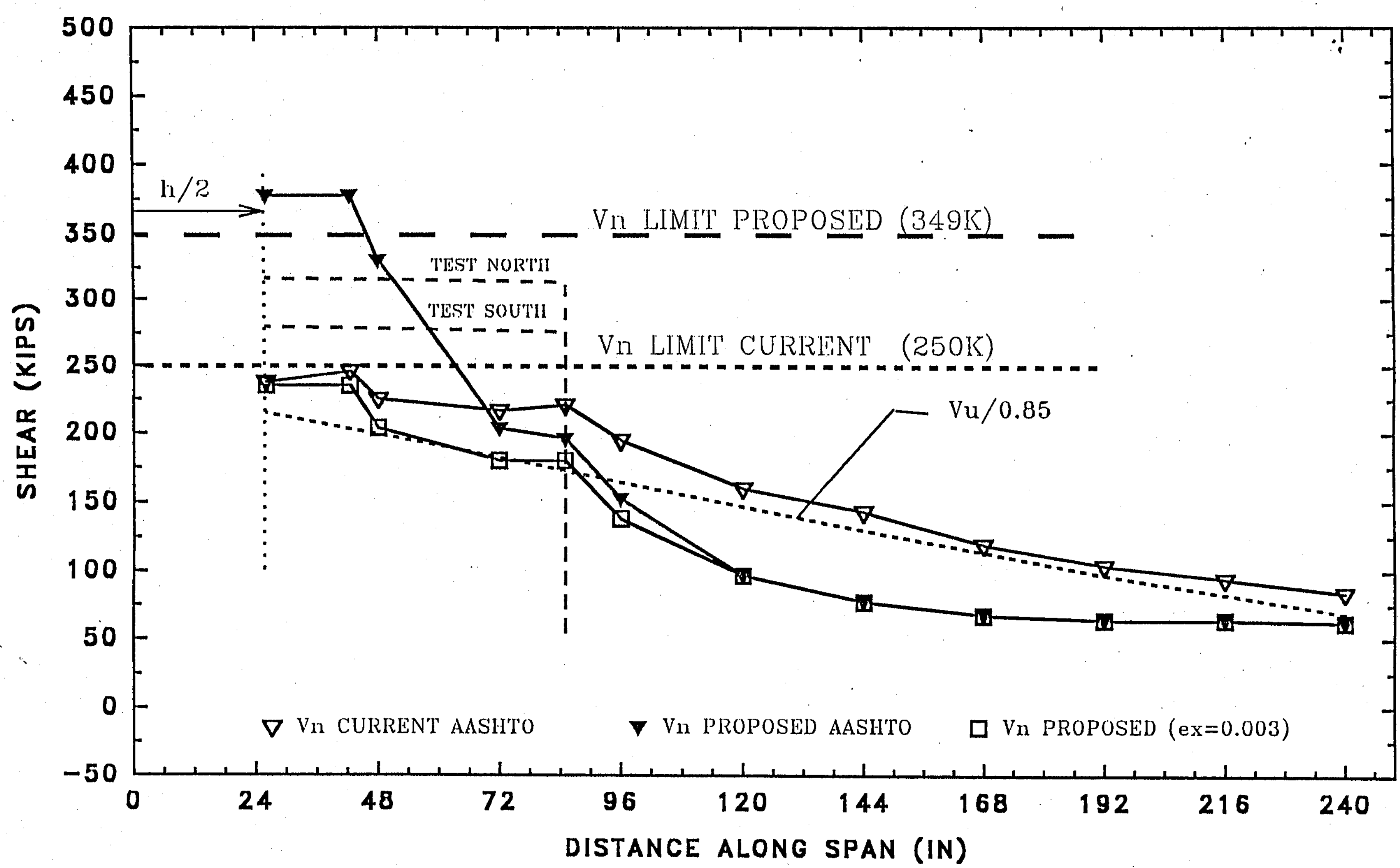
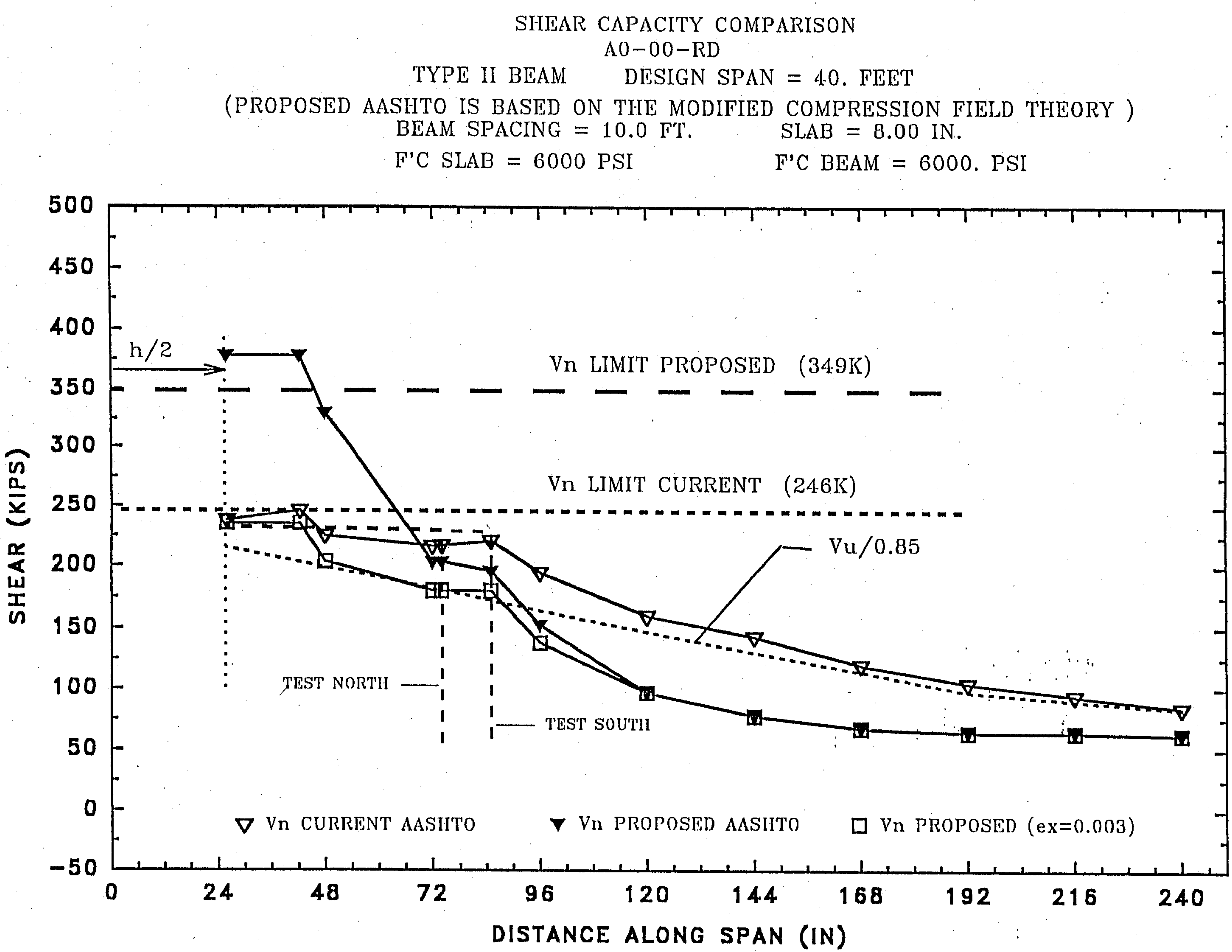


FIGURE A4-1



SHEAR CAPACITY COMPARISON
A0-00-RD
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI
SHEAR (KIPS)
500
450
400
350
300
250
200
150
100
50
0
-50
h/2
Vn LIMIT PROPOSED (349K)
Vn LIMIT CURRENT (246K)
Vu/0.85
TEST NORTH
TEST SOUTH
Vn CURRENT AASHTO
Vn PROPOSED AASHTO
Vn PROPOSED (ex=0.003)
0
24
48
72
96
120
144
168
192
216
240
DISTANCE ALONG SPAN (IN)

FIGURE A4-2

SHEAR CAPACITY COMPARISON
A0-25-R
TYPE II BEAM   DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT.   SLAB = 8.00 IN.
F'C SLAB = 6000 PSI   F'C BEAM = 6000. PSI

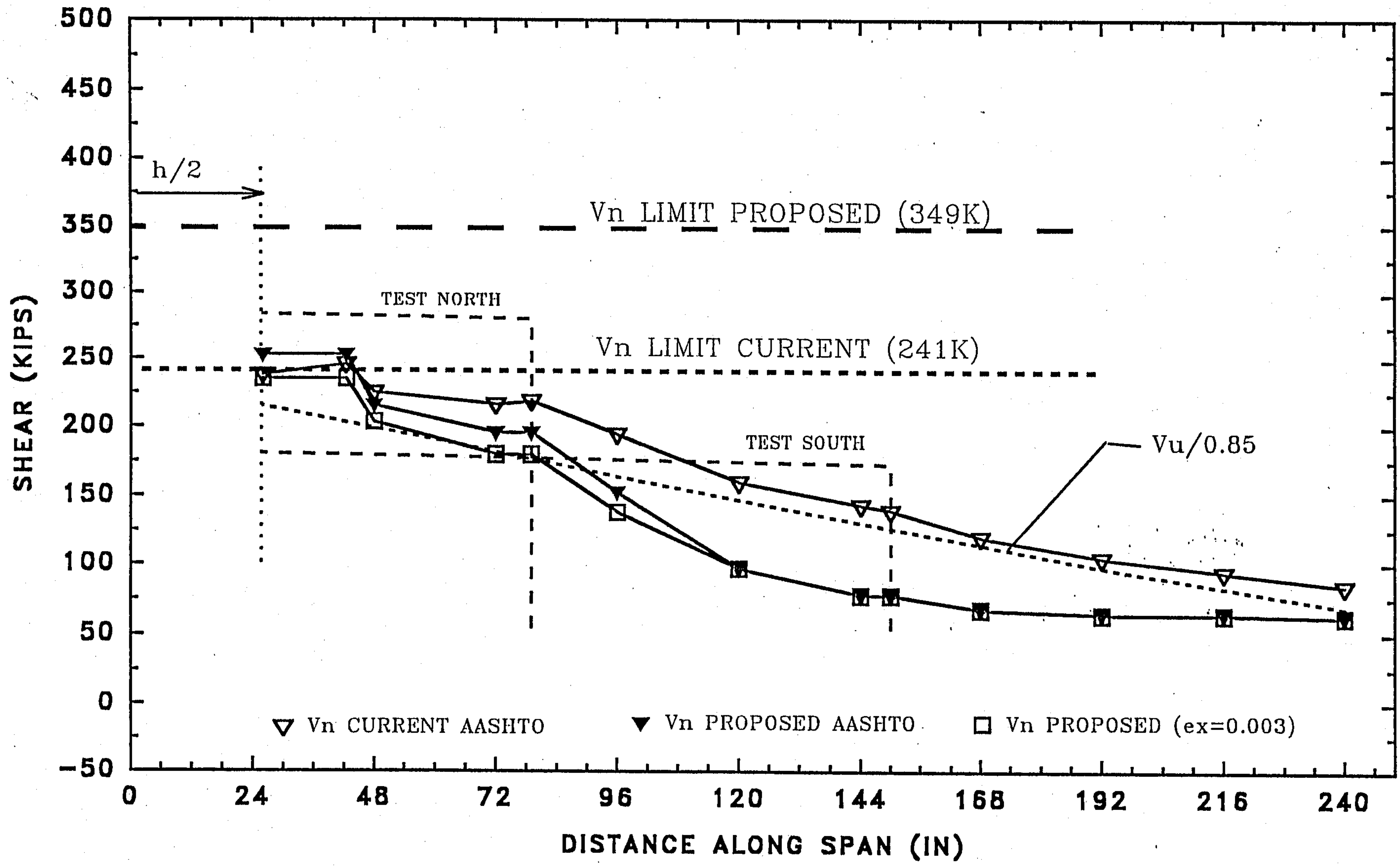


FIGURE A4-3

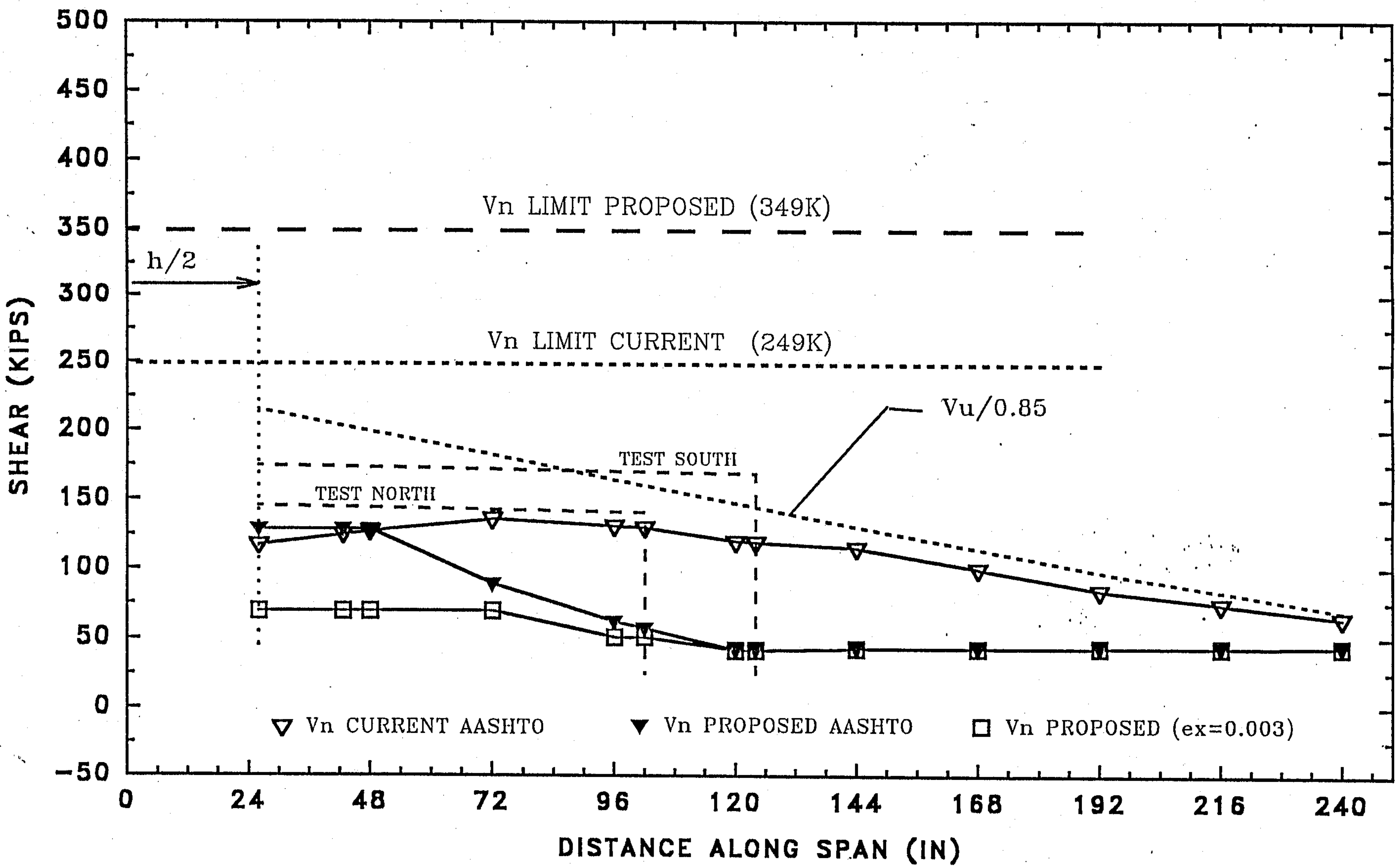

SHEAR CAPACITY COMPARISON
A1-00-M
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI
SHEAR (KIPS)
500
450
400
350
300
250
200
150
100
50
0
-50
Vn LIMIT PROPOSED (349K)
h/2
Vn LIMIT CURRENT (249K)
Vu/0.85
TEST SOUTH
TEST NORTH
Vn CURRENT AASHTO
Vn PROPOSED AASHTO
Vn PROPOSED (ex=0.003)
0
24
48
72
96
120
144
168
192
216
240
DISTANCE ALONG SPAN (IN)

FIGURE A4-4

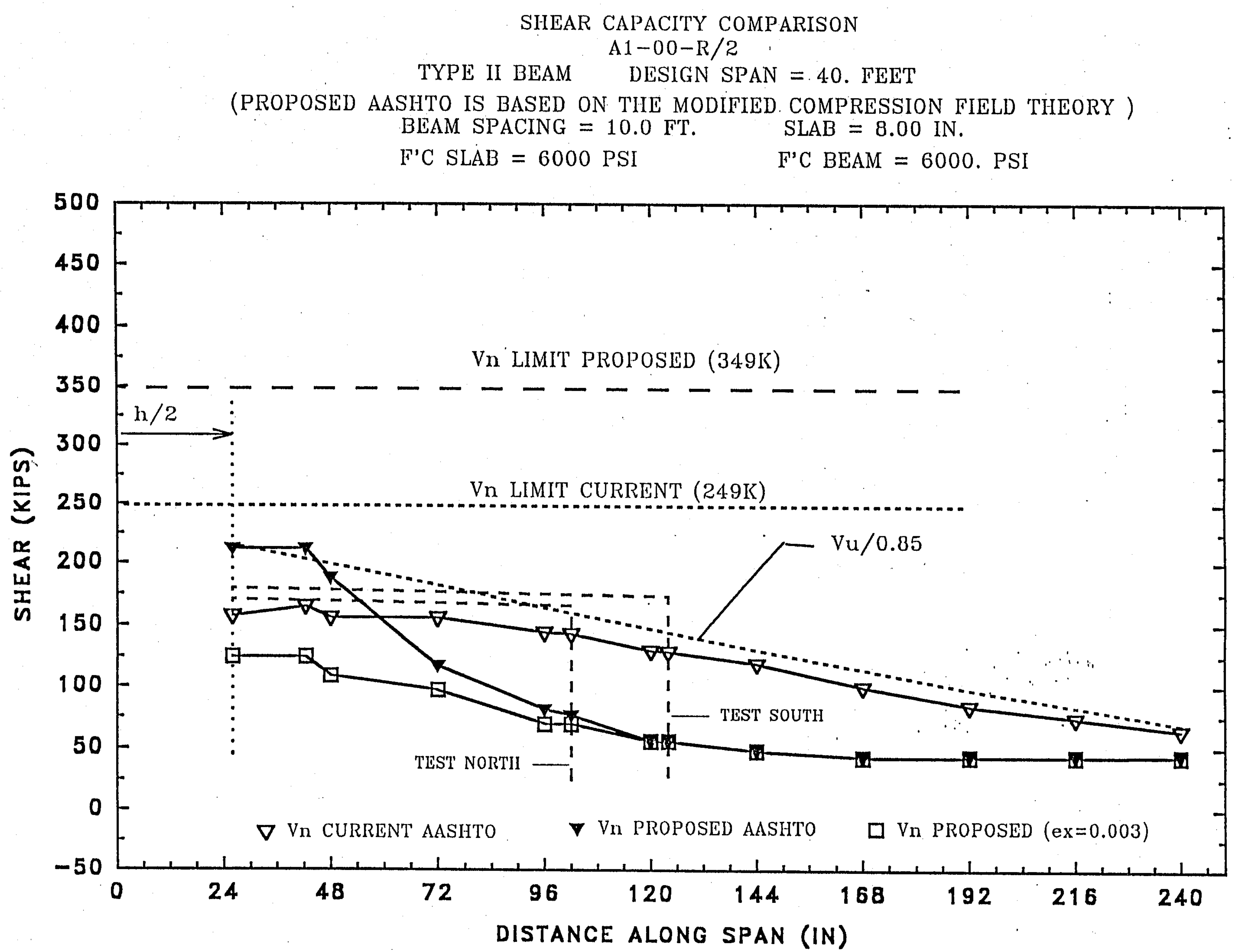

SHEAR CAPACITY COMPARISON
A1-00-R/2
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI
Vn LIMIT PROPOSED (349K)
Vn LIMIT CURRENT (249K)
h/2
Vu/0.85
TEST SOUTH
TEST NORTH
Vn CURRENT AASHTO
Vn PROPOSED AASHTO
Vn PROPOSED (ex=0.003)
SHEAR (KIPS)
DISTANCE ALONG SPAN (IN)
500
450
400
350
300
250
200
150
100
50
0
-50
0
24
48
72
96
120
144
168
192
216
240

FIGURE A4-5

SHEAR CAPACITY COMPARISON
A1-00-R
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

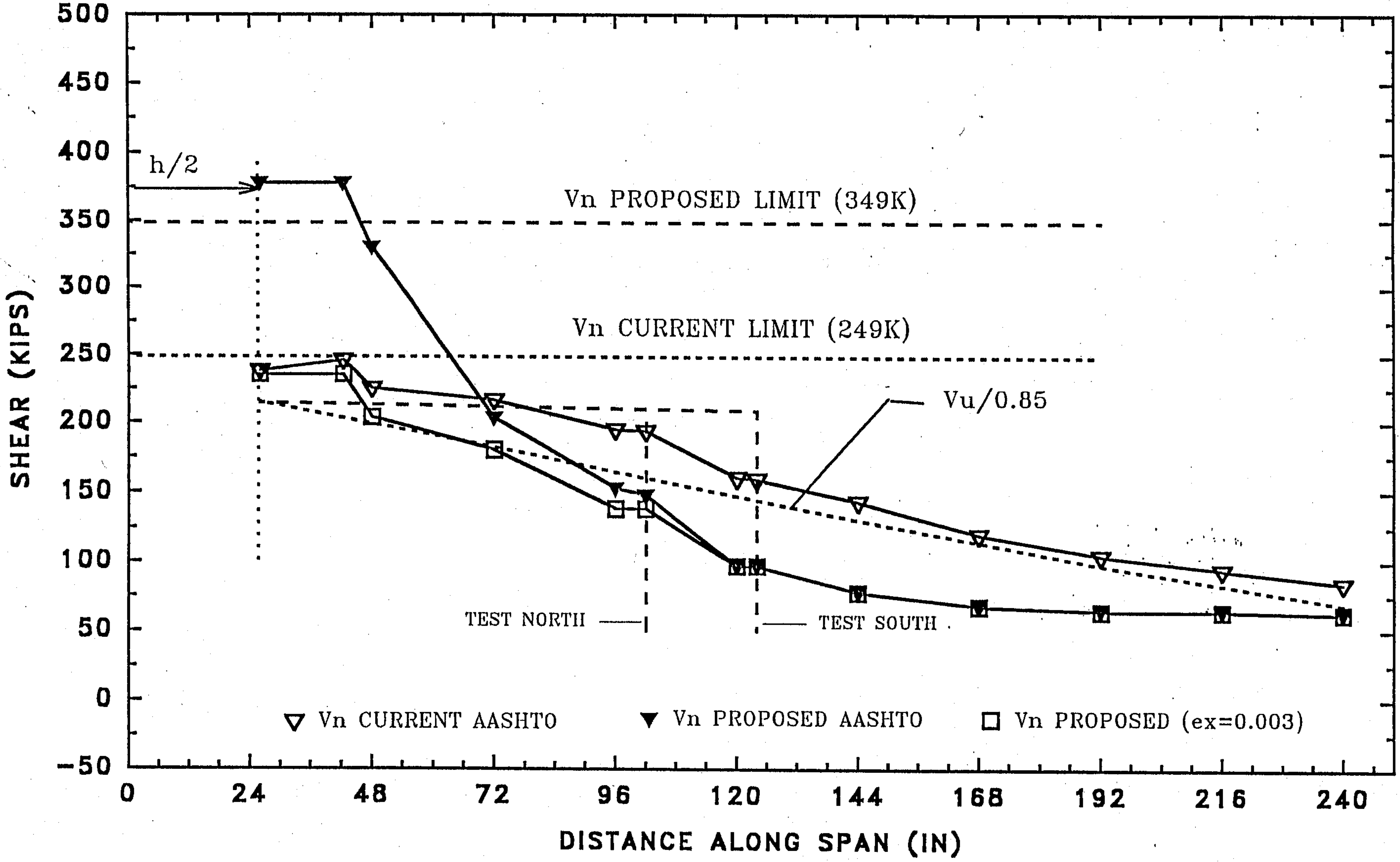


FIGURE A4-6

SHEAR CAPACITY COMPARISON
A1-00-RD
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

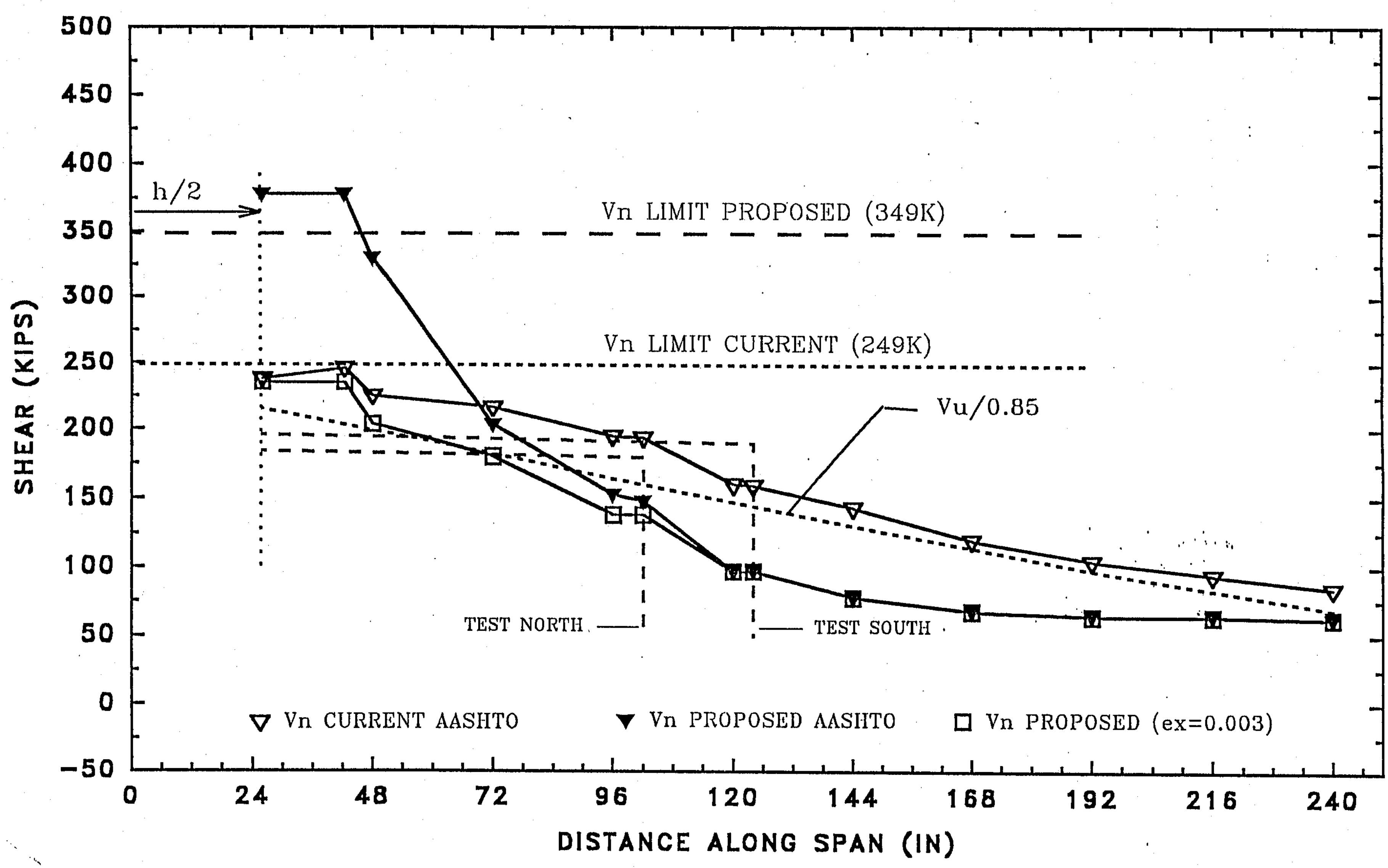


FIGURE A4-7

SHEAR CAPACITY COMPARISON
A1-00-3R/2
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

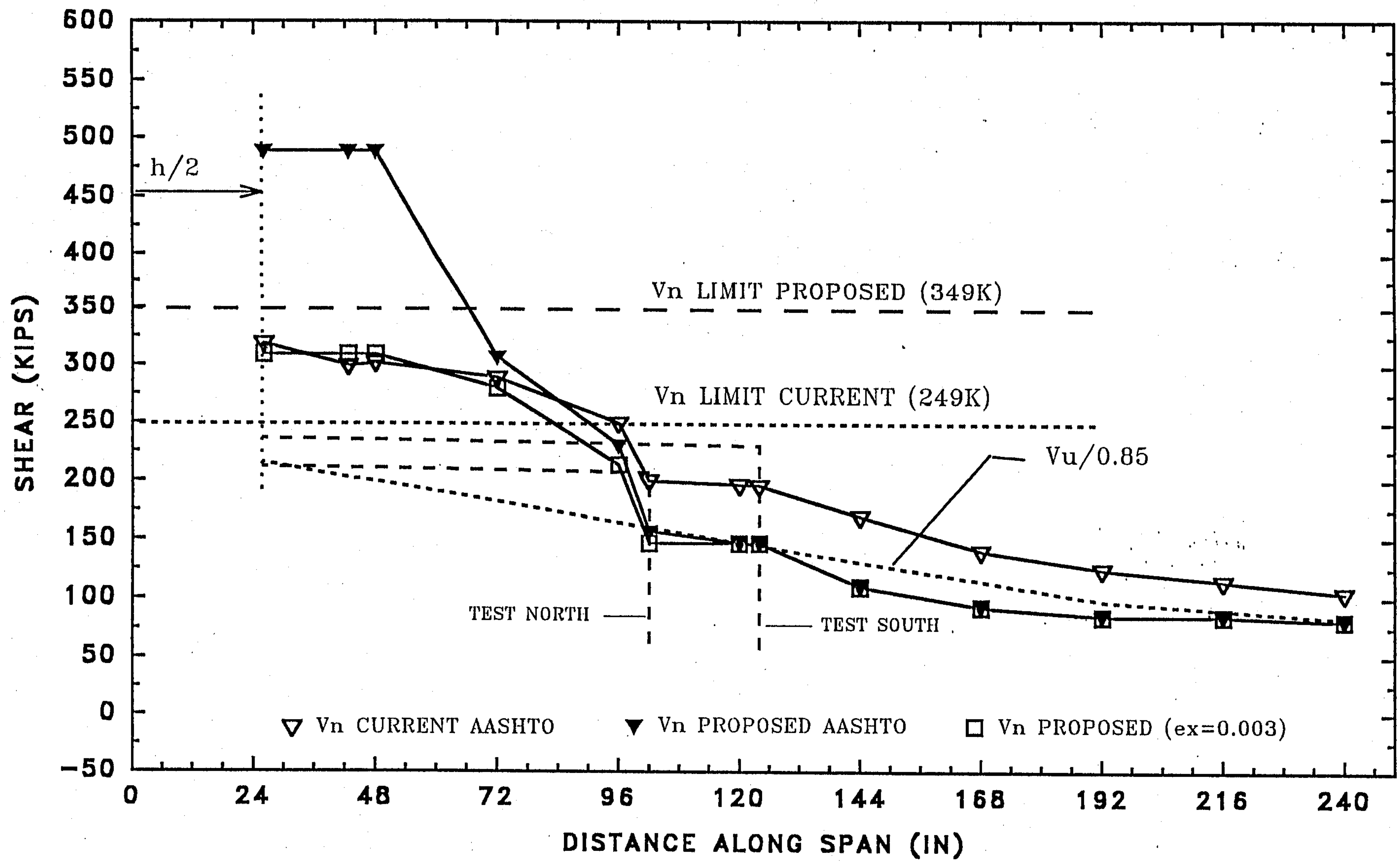


FIGURE A4-8

SHEAR CAPACITY COMPARISON
A2-00-2R
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

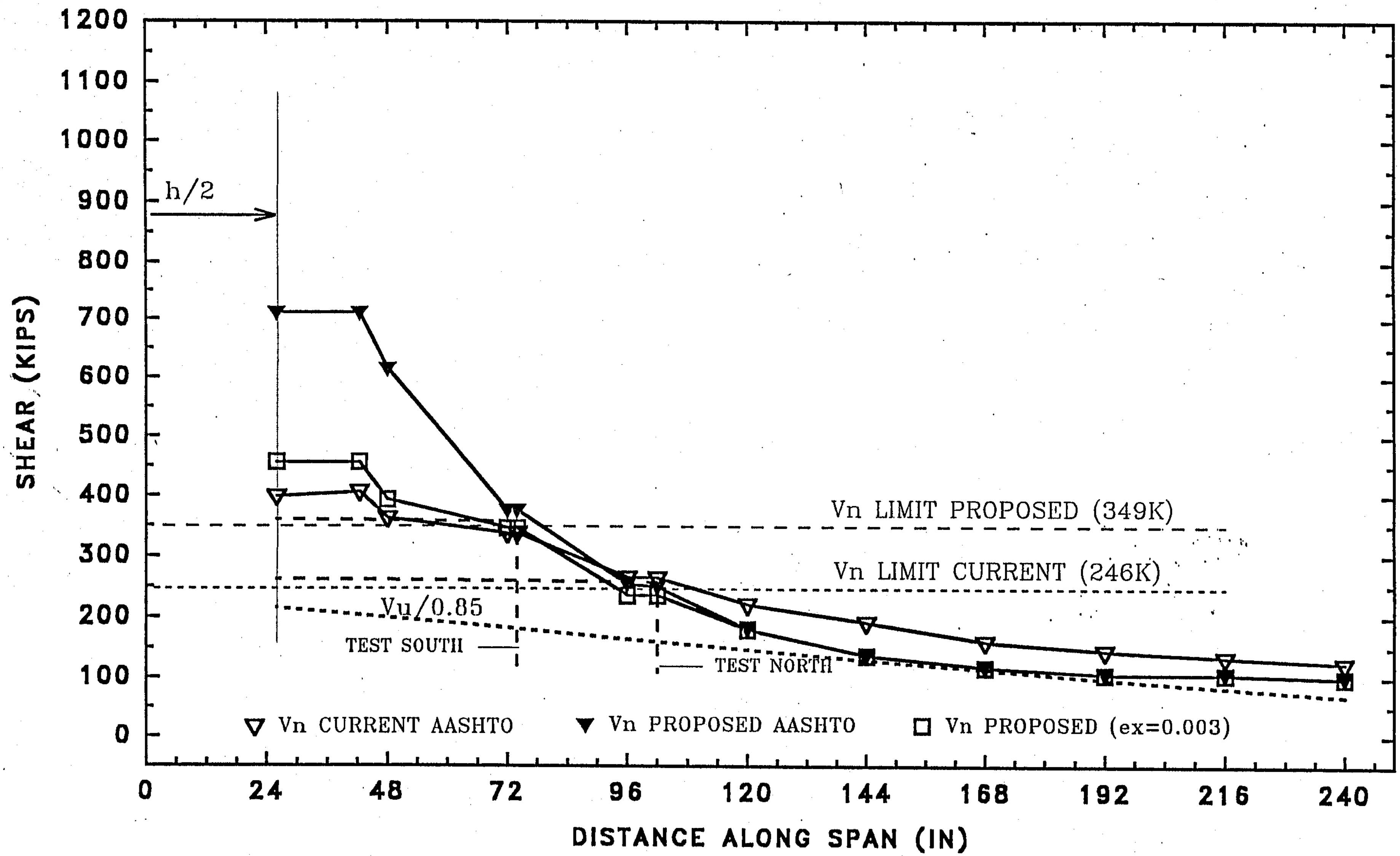


FIGURE A4-9

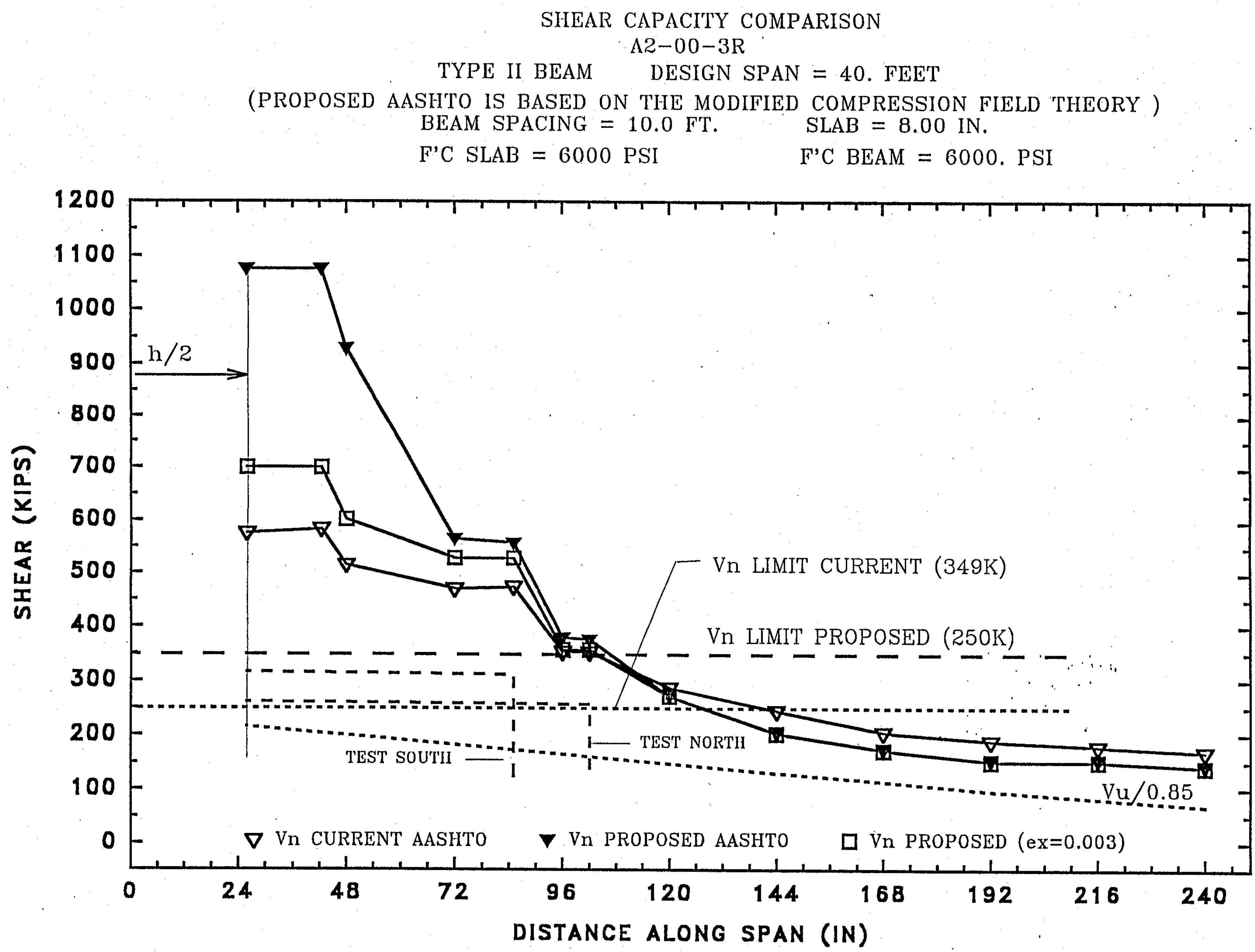

SHEAR CAPACITY COMPARISON
A2-00-3R
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI
SHEAR (KIPS)
1200
1100
1000
900
800
700
600
500
400
300
200
100
0
h/2
Vn LIMIT CURRENT (349K)
Vn LIMIT PROPOSED (250K)
TEST NORTH
TEST SOUTH
Vu/0.85
Vn CURRENT AASHTO
Vn PROPOSED AASHTO
Vn PROPOSED (ex=0.003)
0
24
48
72
96
120
144
168
192
216
240
DISTANCE ALONG SPAN (IN)

FIGURE A4-10

SHEAR CAPACITY COMPARISON
A2-00-3RD
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

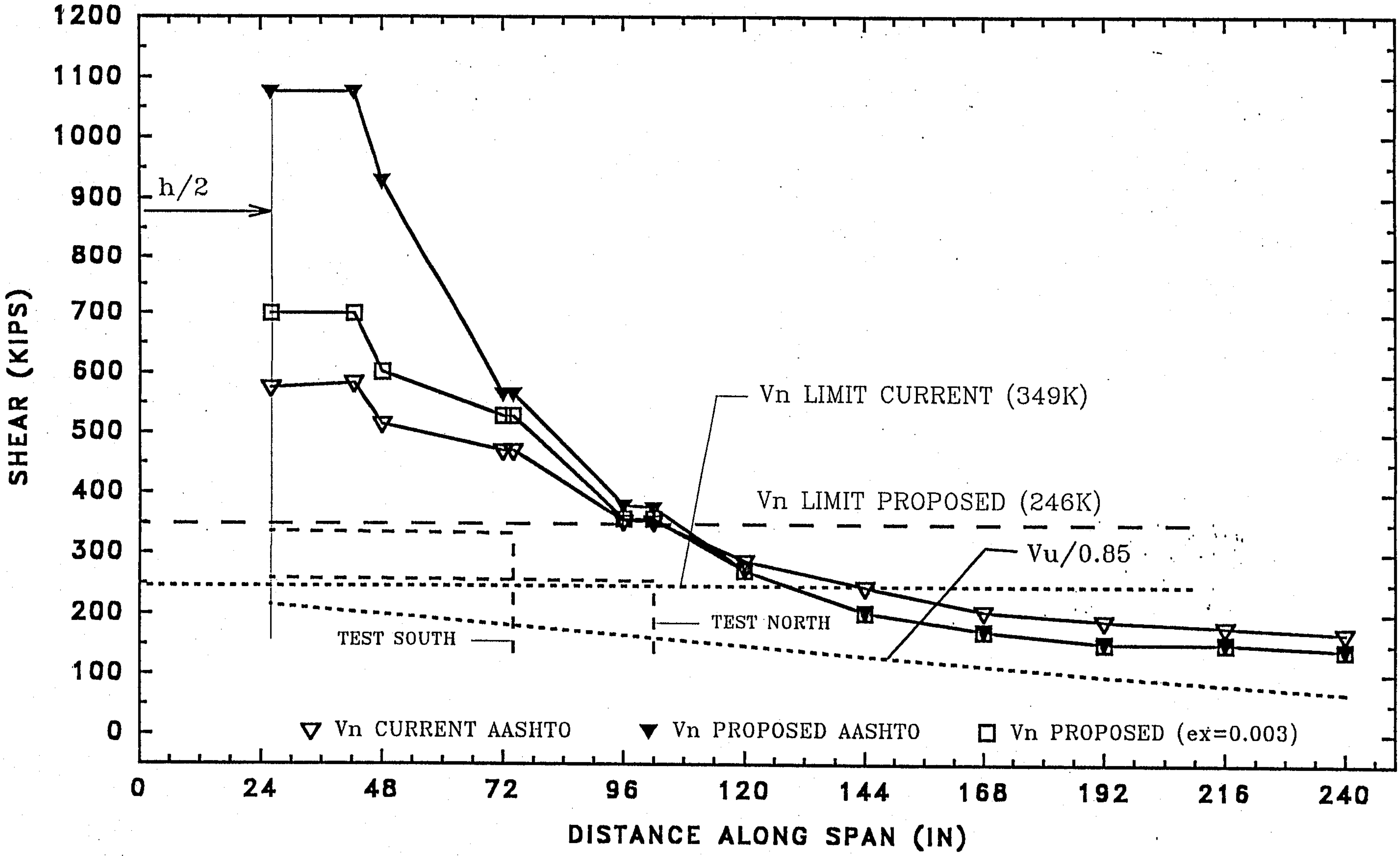


FIGURE A4-11

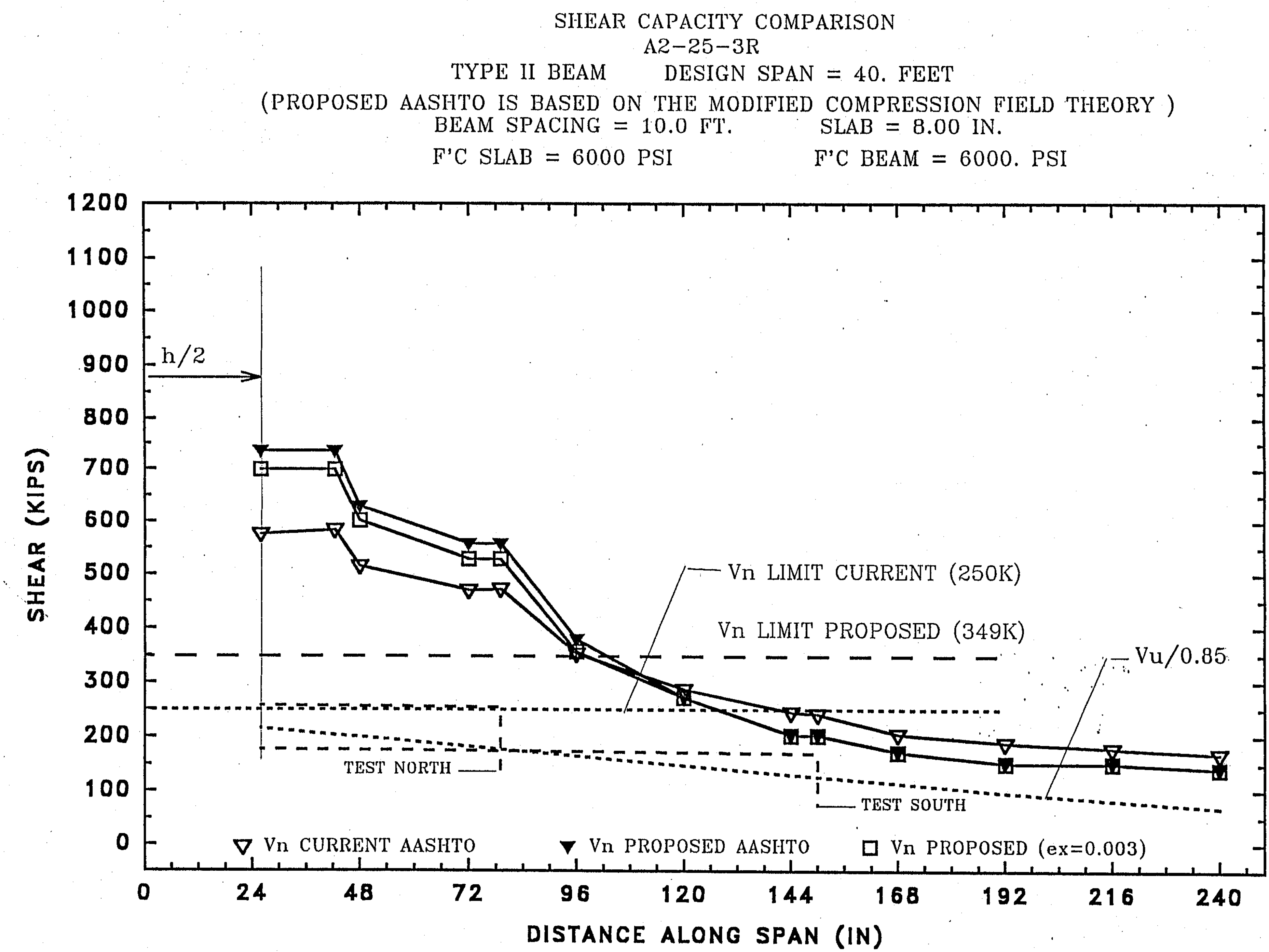

SHEAR CAPACITY COMPARISON
A2-25-3R
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI
h/2
Vn LIMIT CURRENT (250K)
Vn LIMIT PROPOSED (349K)
Vu/0.85
TEST NORTH
TEST SOUTH
Vn CURRENT AASHTO
Vn PROPOSED AASHTO
Vn PROPOSED (ex=0.003)
SHEAR (KIPS)
1200
1100
1000
900
800
700
600
500
400
300
200
100
0
0
24
48
72
96
120
144
168
192
216
240
DISTANCE ALONG SPAN (IN)

FIGURE A4-12

SHEAR CAPACITY COMPARISON
A2-50-3R
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

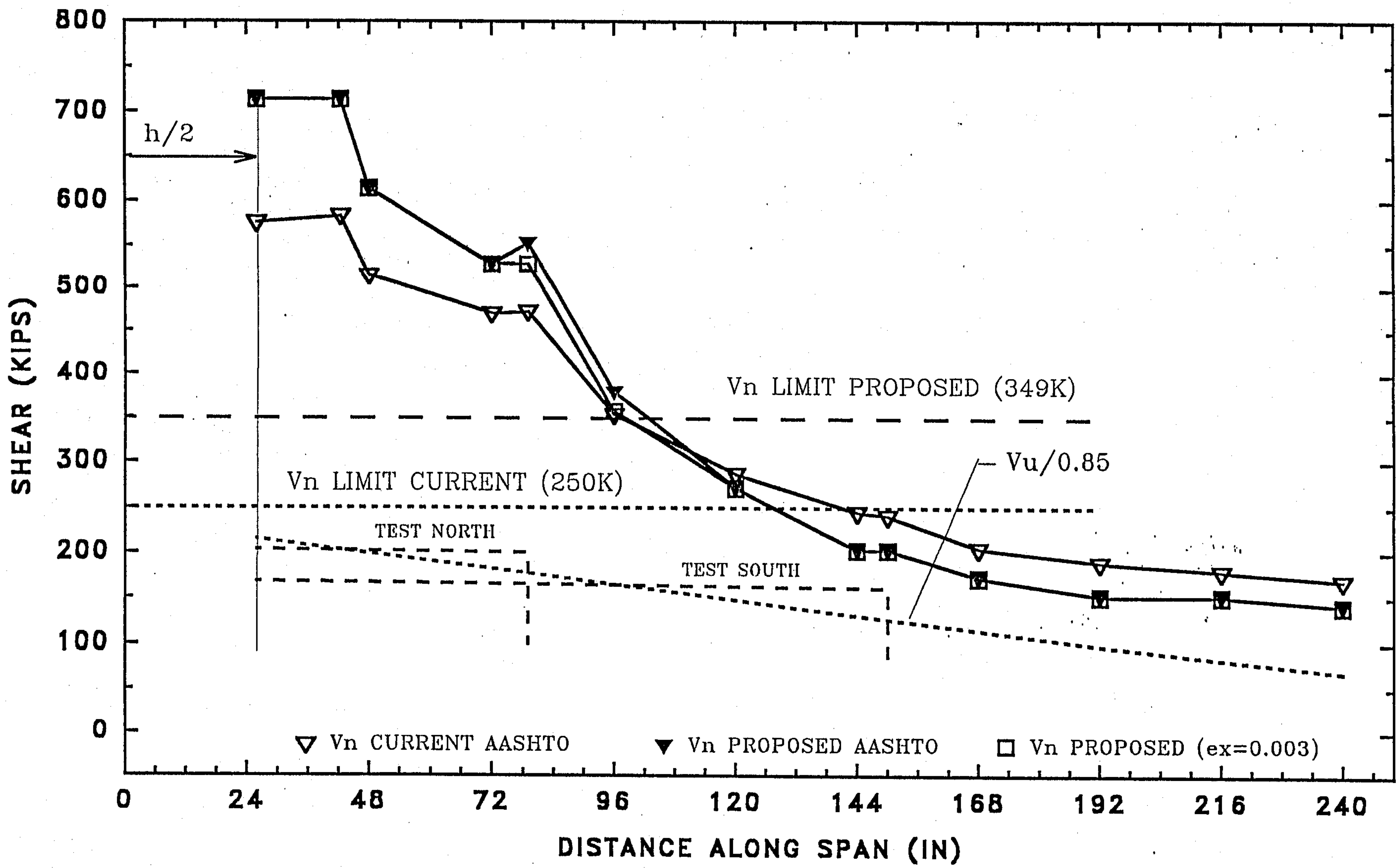
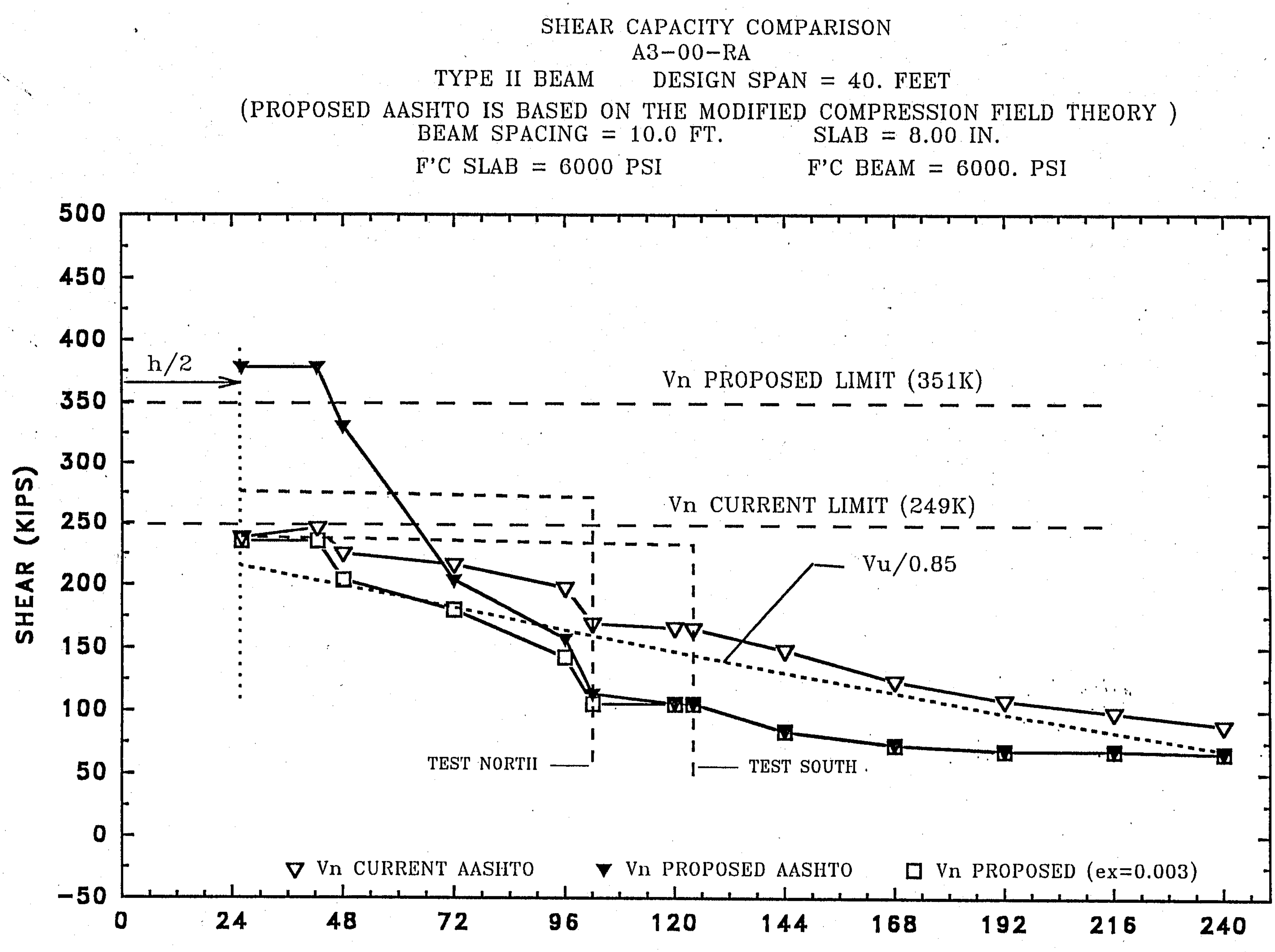


FIGURE A4-13


SHEAR CAPACITY COMPARISON
A3-00-RA
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI
SHEAR (KIPS)
500
450
400
350
300
250
200
150
100
50
0
-50
h/2
Vn PROPOSED LIMIT (351K)
Vn CURRENT LIMIT (249K)
Vu/0.85
TEST NORTH
TEST SOUTH
Vn CURRENT AASHTO
Vn PROPOSED AASHTO
Vn PROPOSED (ex=0.003)
0
24
48
72
96
120
144
168
192
216
240
DISTANCE ALONG SPAN (IN)
FIGURE A4-14

SHEAR CAPACITY COMPARISON
A3-00-RB
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

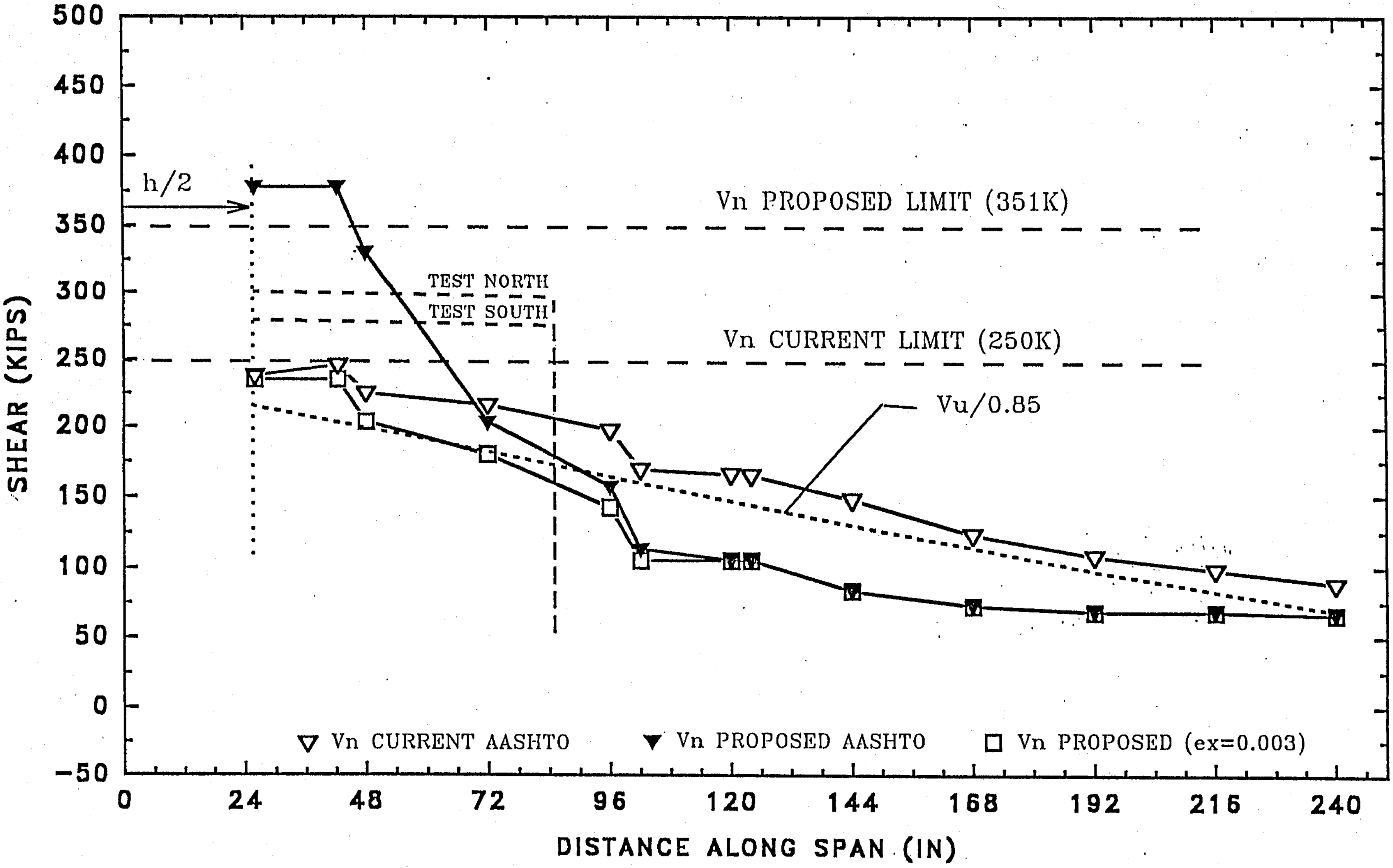


FIGURE A4-15

SHEAR CAPACITY COMPARISON
B0-00-R
TYPE II BEAM   DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT.   SLAB = 8.00 IN.
F'C SLAB = 6000 PSI   F'C BEAM = 6000. PSI

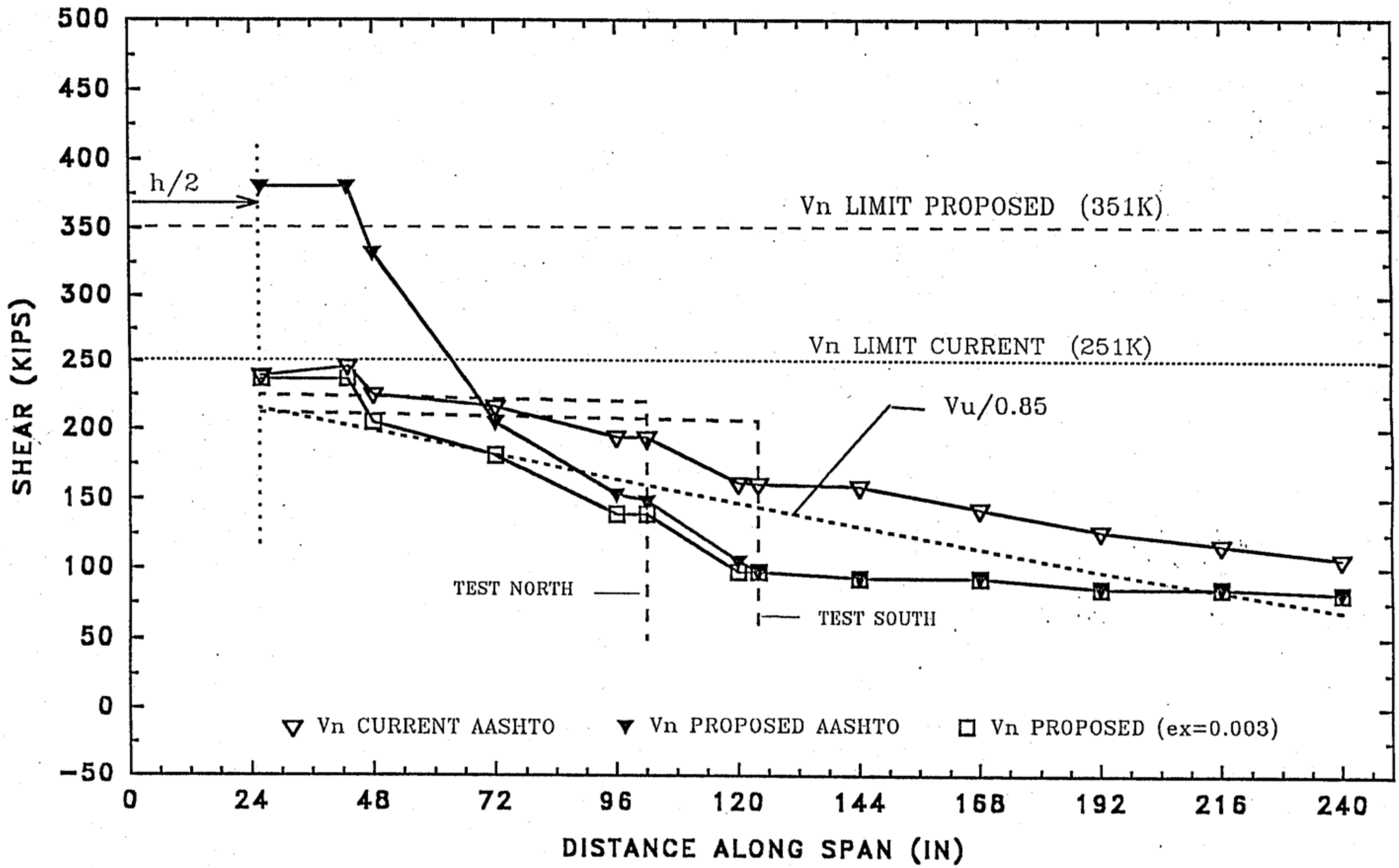


FIGURE A4-16

SHEAR CAPACITY COMPARISON
B0-00-2R
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

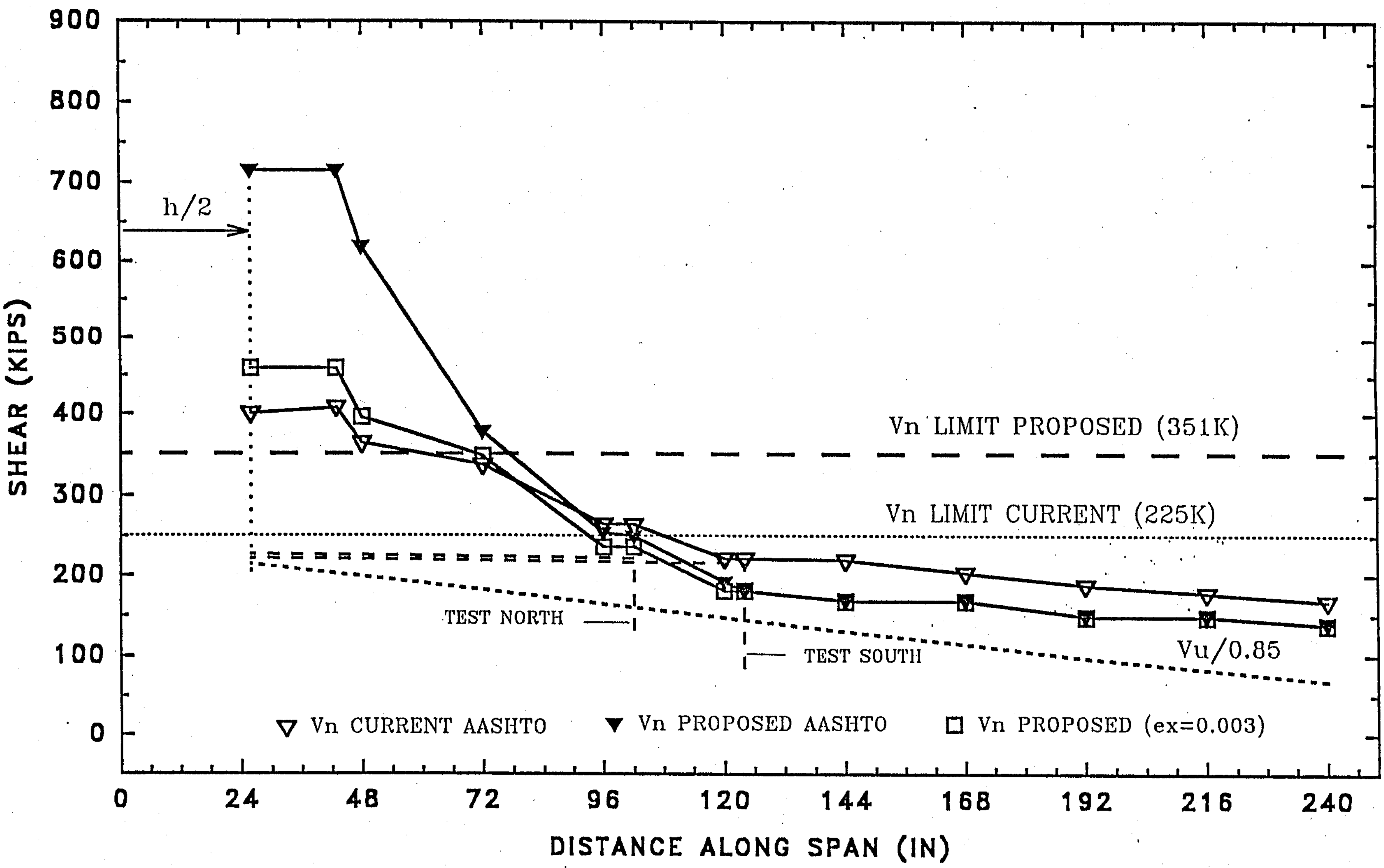
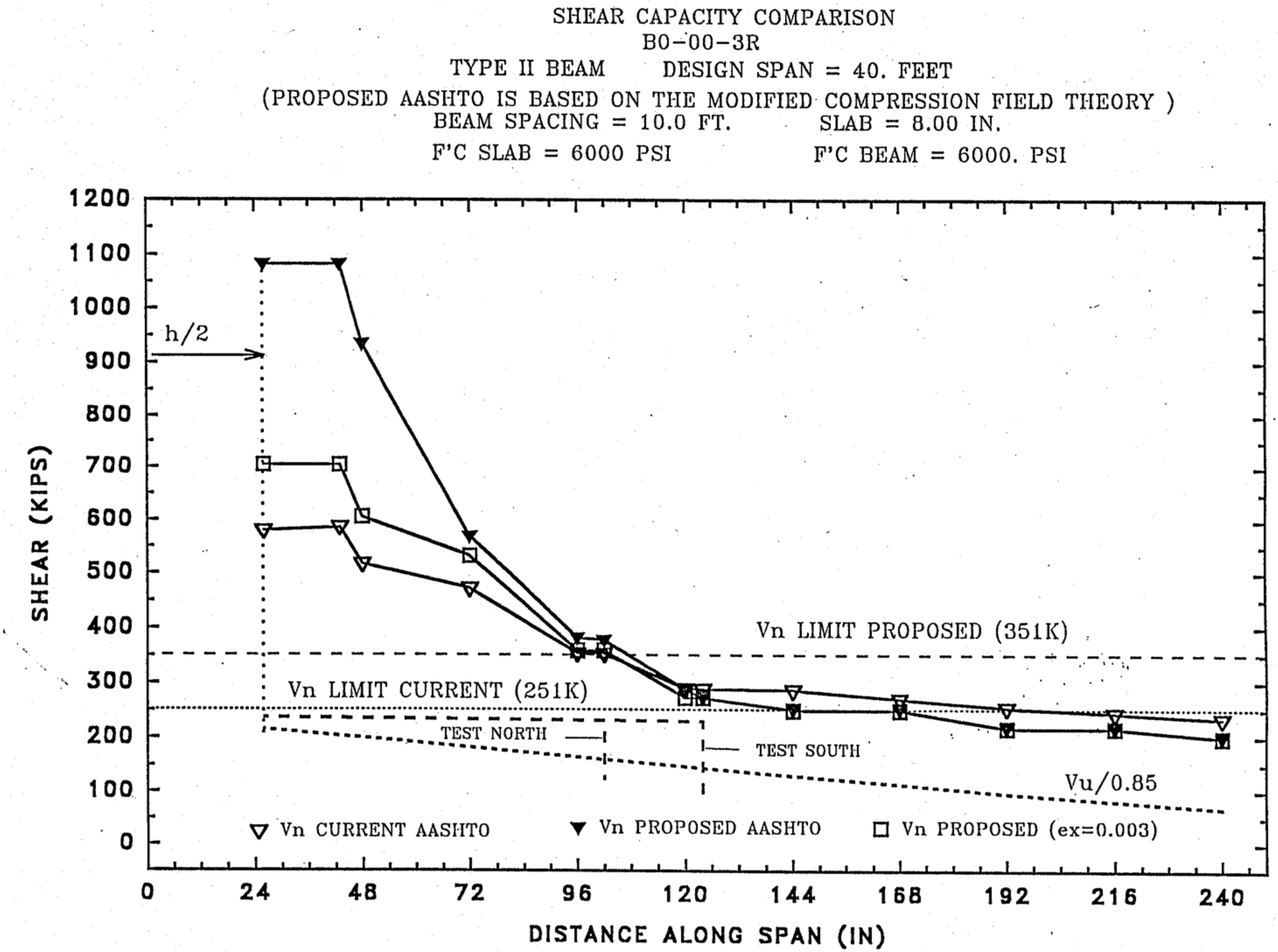


FIGURE A4-17



SHEAR CAPACITY COMPARISON
B0-00-3R
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI
SHEAR (KIPS)
DISTANCE ALONG SPAN (IN)
h/2
Vn LIMIT PROPOSED (351K)
Vn LIMIT CURRENT (251K)
TEST NORTH
TEST SOUTH
Vu/0.85
Vn CURRENT AASHTO
Vn PROPOSED AASHTO
Vn PROPOSED (ex=0.003)

FIGURE A4-18

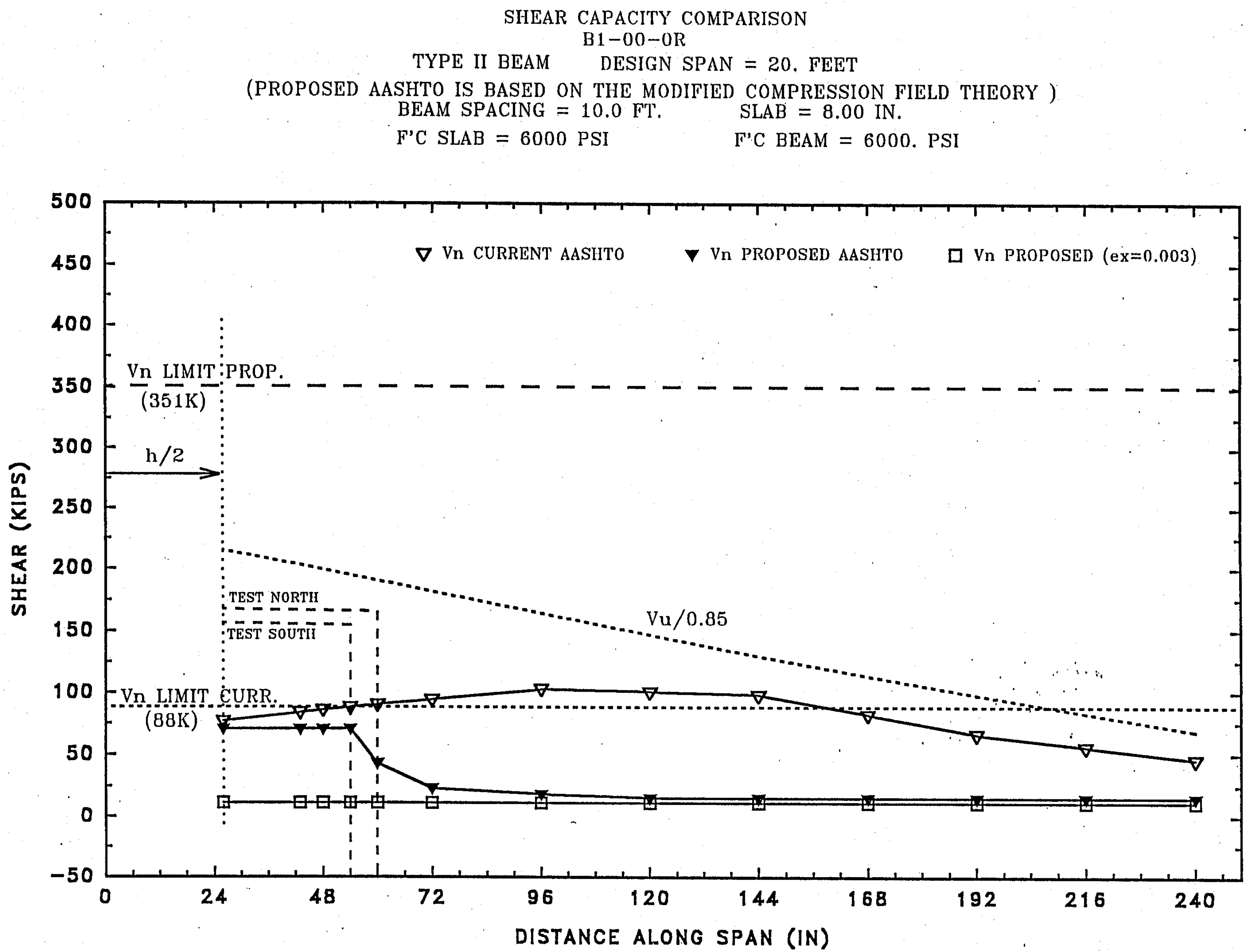

SHEAR CAPACITY COMPARISON
B1-00-0R
TYPE II BEAM DESIGN SPAN = 20. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI
Vn CURRENT AASHTO
Vn PROPOSED AASHTO
Vn PROPOSED (ex=0.003)
Vn LIMIT PROP.
(351K)
h/2
TEST NORTH
TEST SOUTH
Vu/0.85
Vn LIMIT CURR.
(88K)
SHEAR (KIPS)
500
450
400
350
300
250
200
150
100
50
0
-50
0
24
48
72
96
120
144
168
192
216
240
DISTANCE ALONG SPAN (IN)

FIGURE A4-19

SHEAR CAPACITY COMPARISON
B1-00-R
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

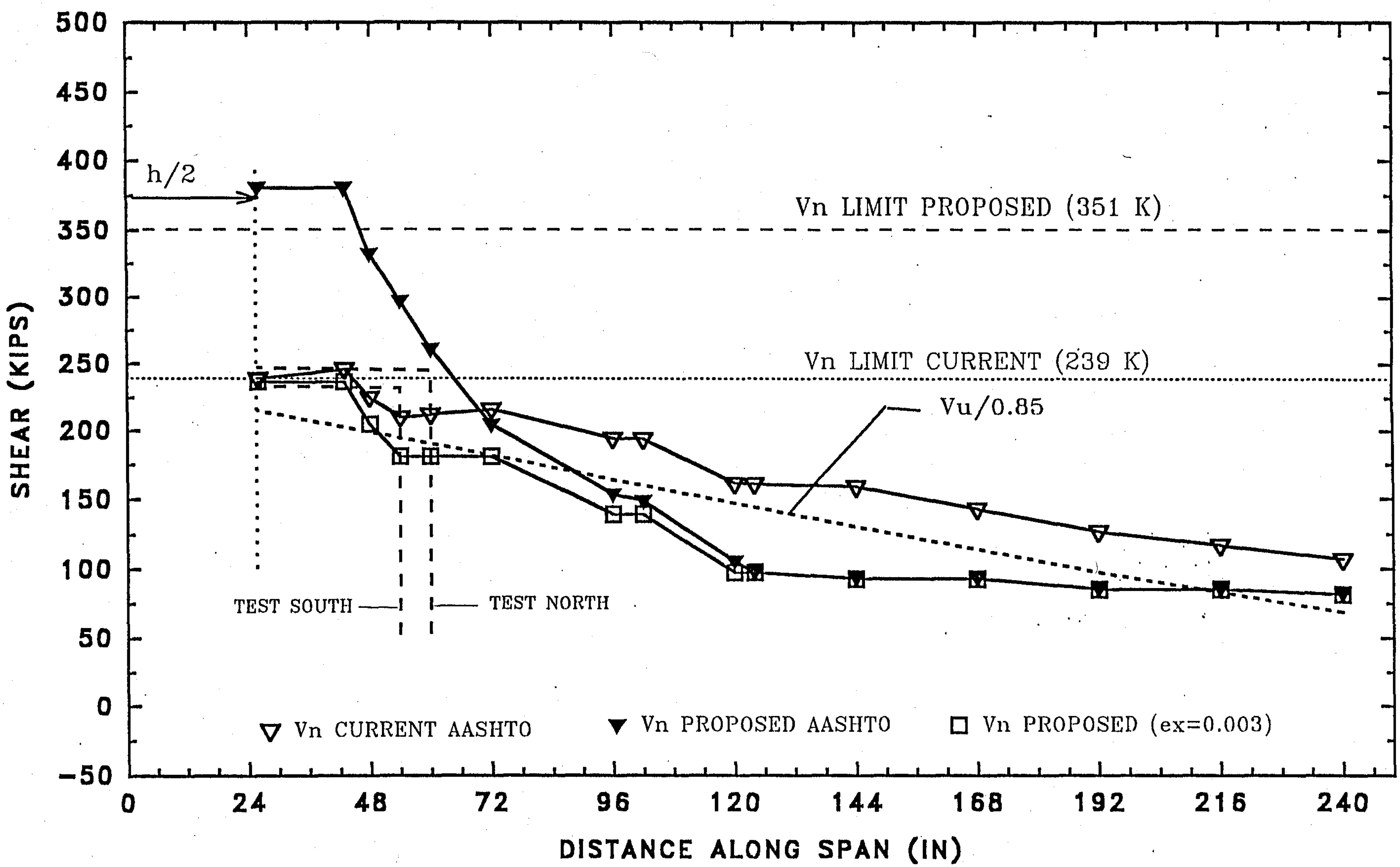


FIGURE A4-20

SHEAR CAPACITY COMPARISON
B1-00-2R
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

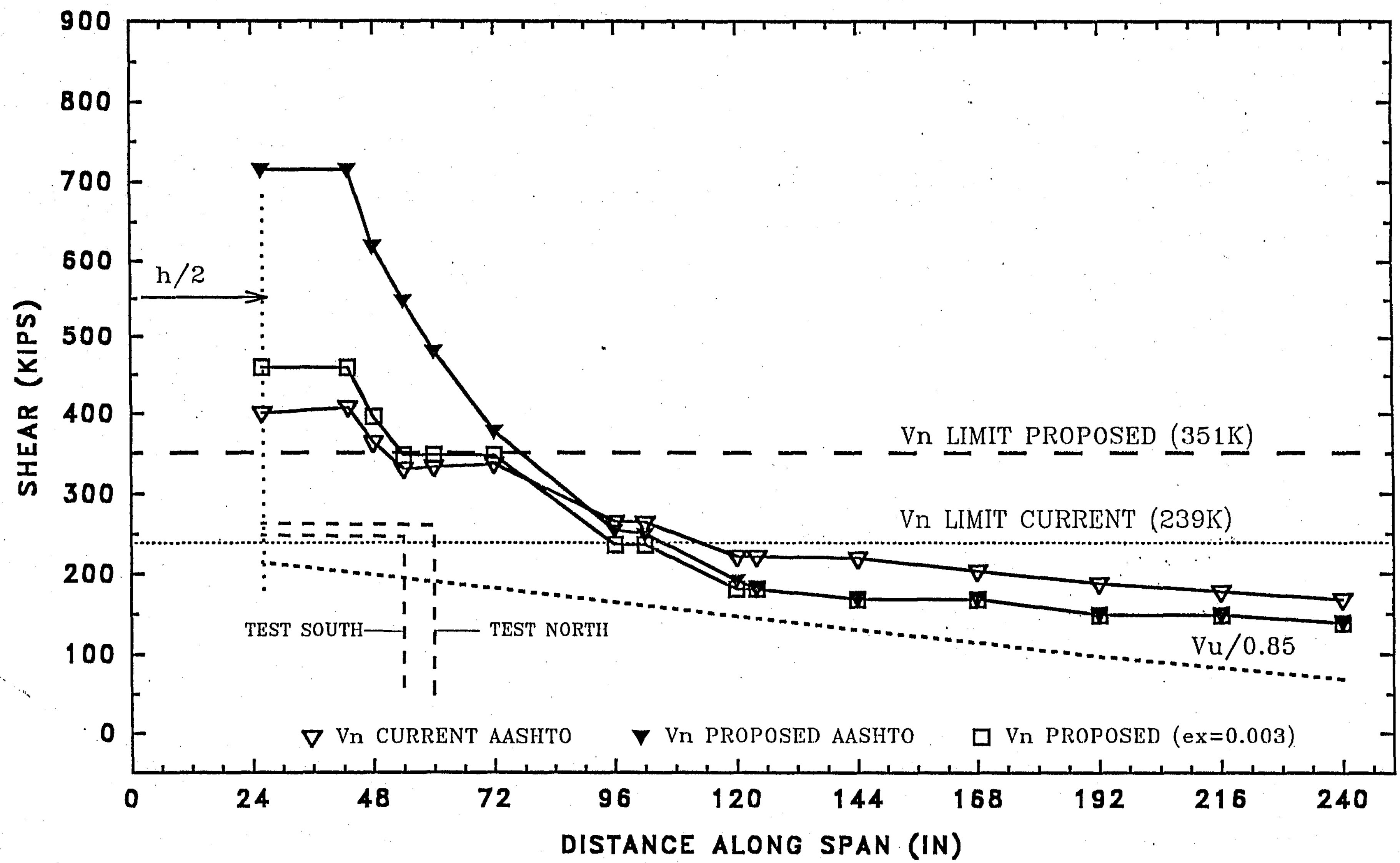


FIGURE A4-21

SHEAR CAPACITY COMPARISON
B1-00-2R #2
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

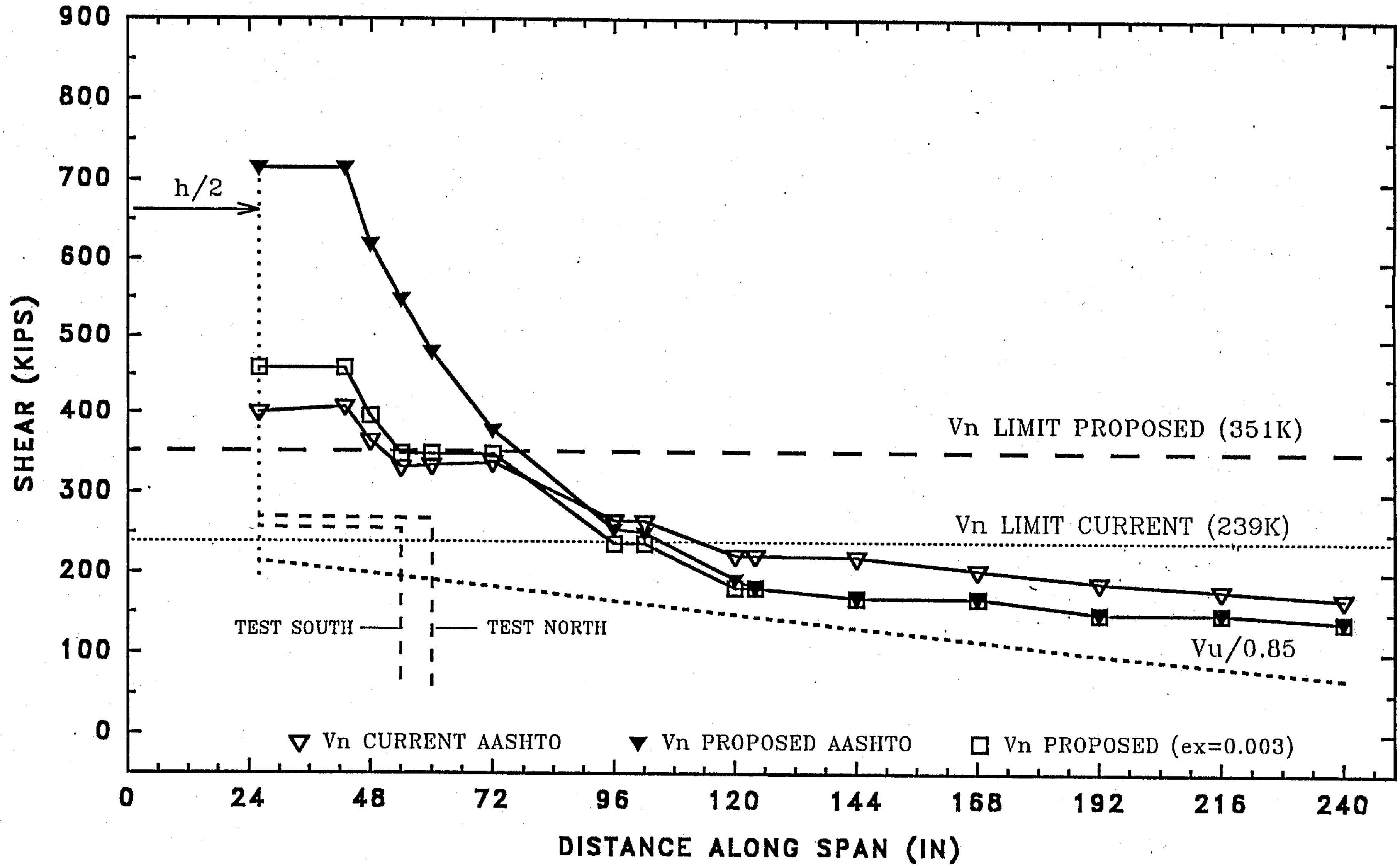


FIGURE A4-22

SHEAR CAPACITY COMPARISON
B1-00-3R
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

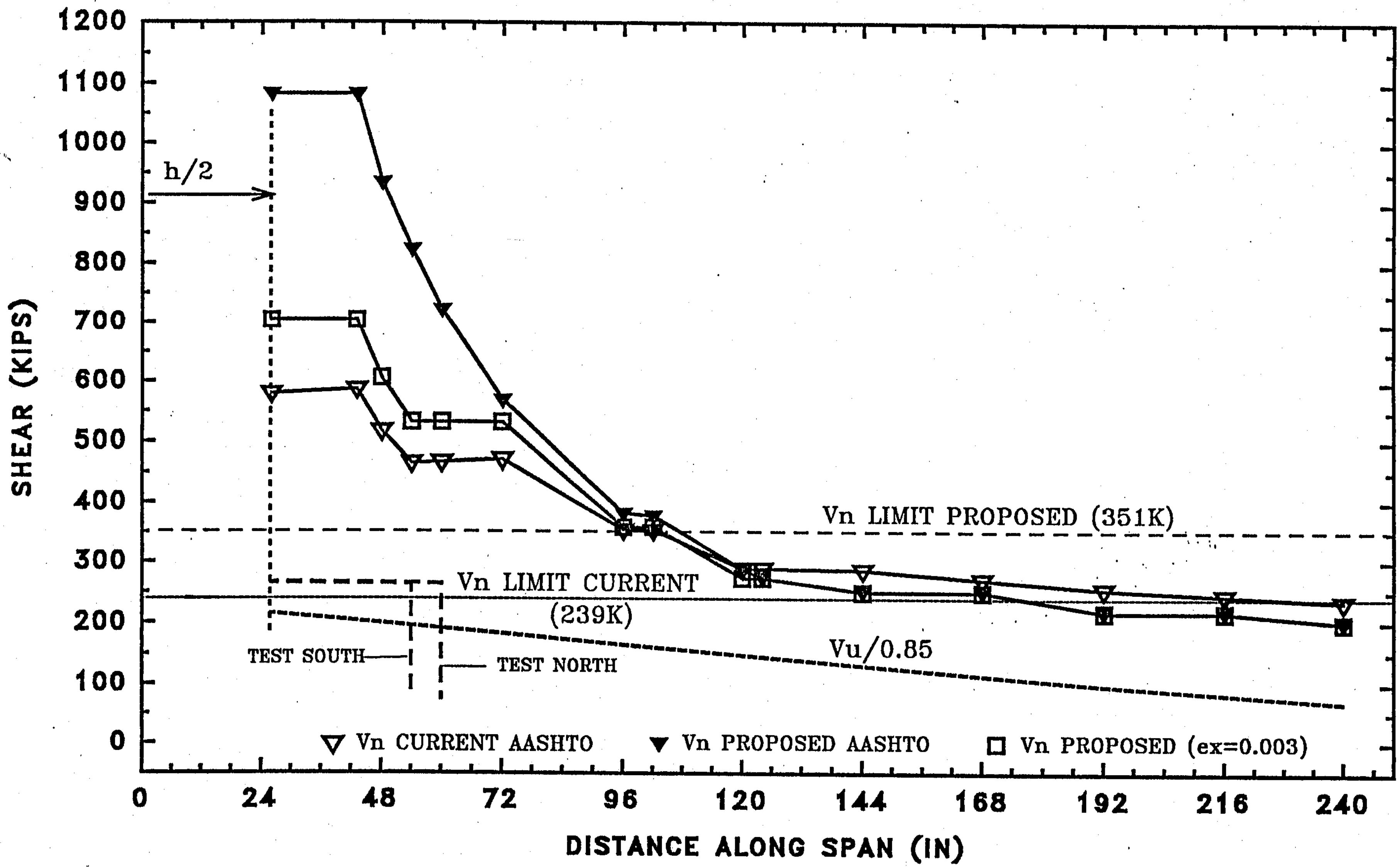


FIGURE A4-23

SHEAR CAPACITY COMPARISON
C0-00-R
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

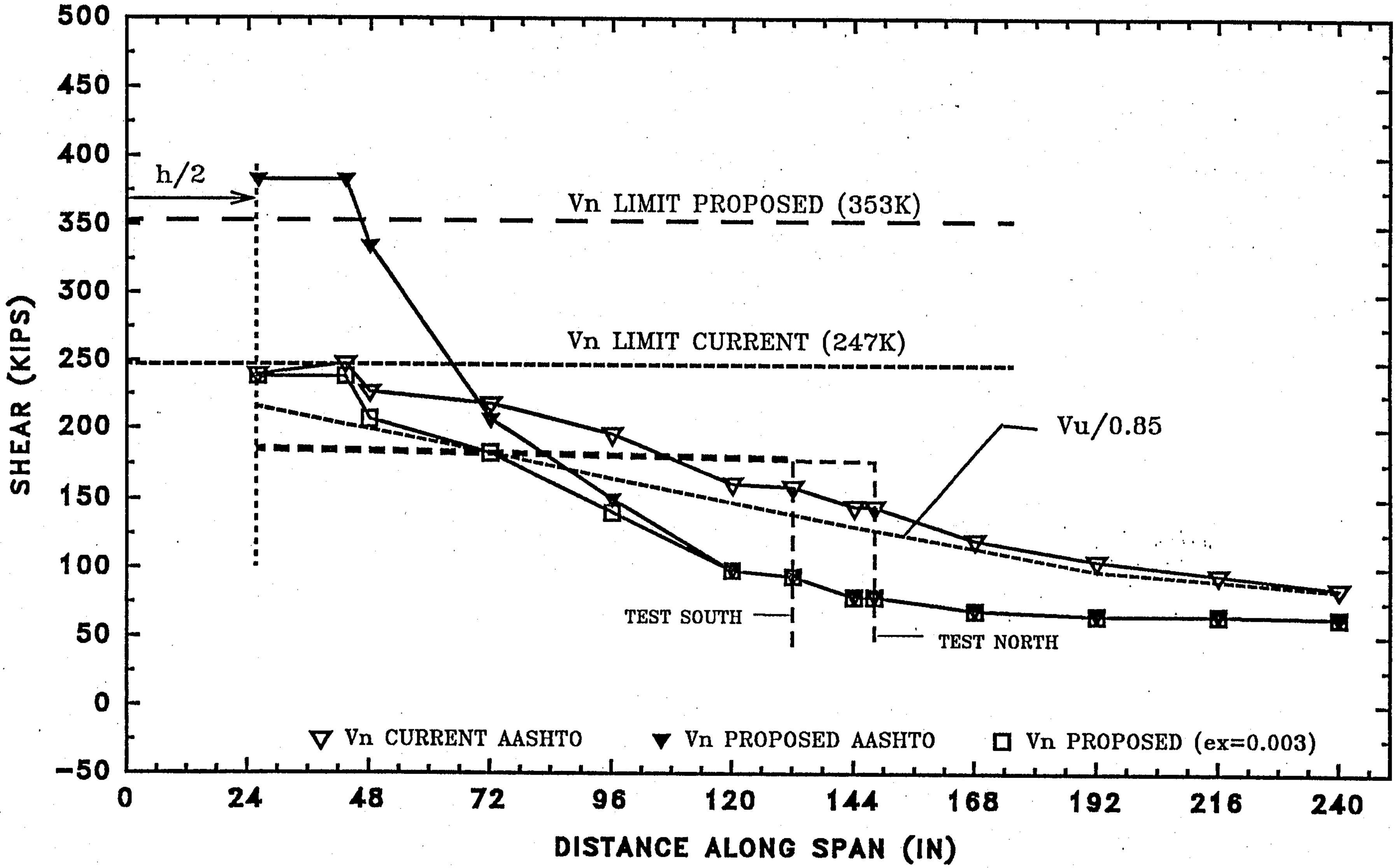


FIGURE A4-24

SHEAR CAPACITY COMPARISON
C0-00-RD
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

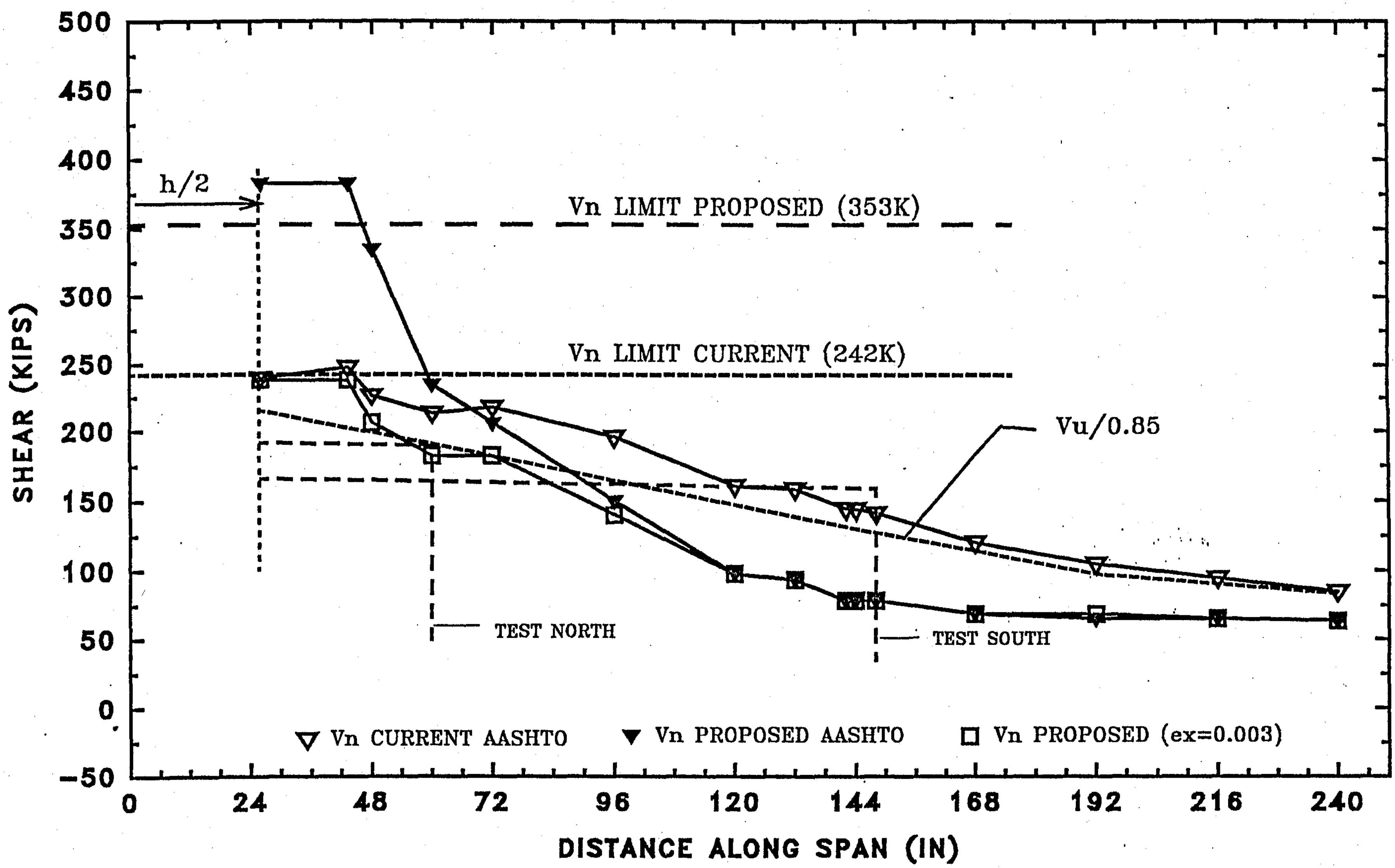


FIGURE A4-25

SHEAR CAPACITY COMPARISON
CO-25-R
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

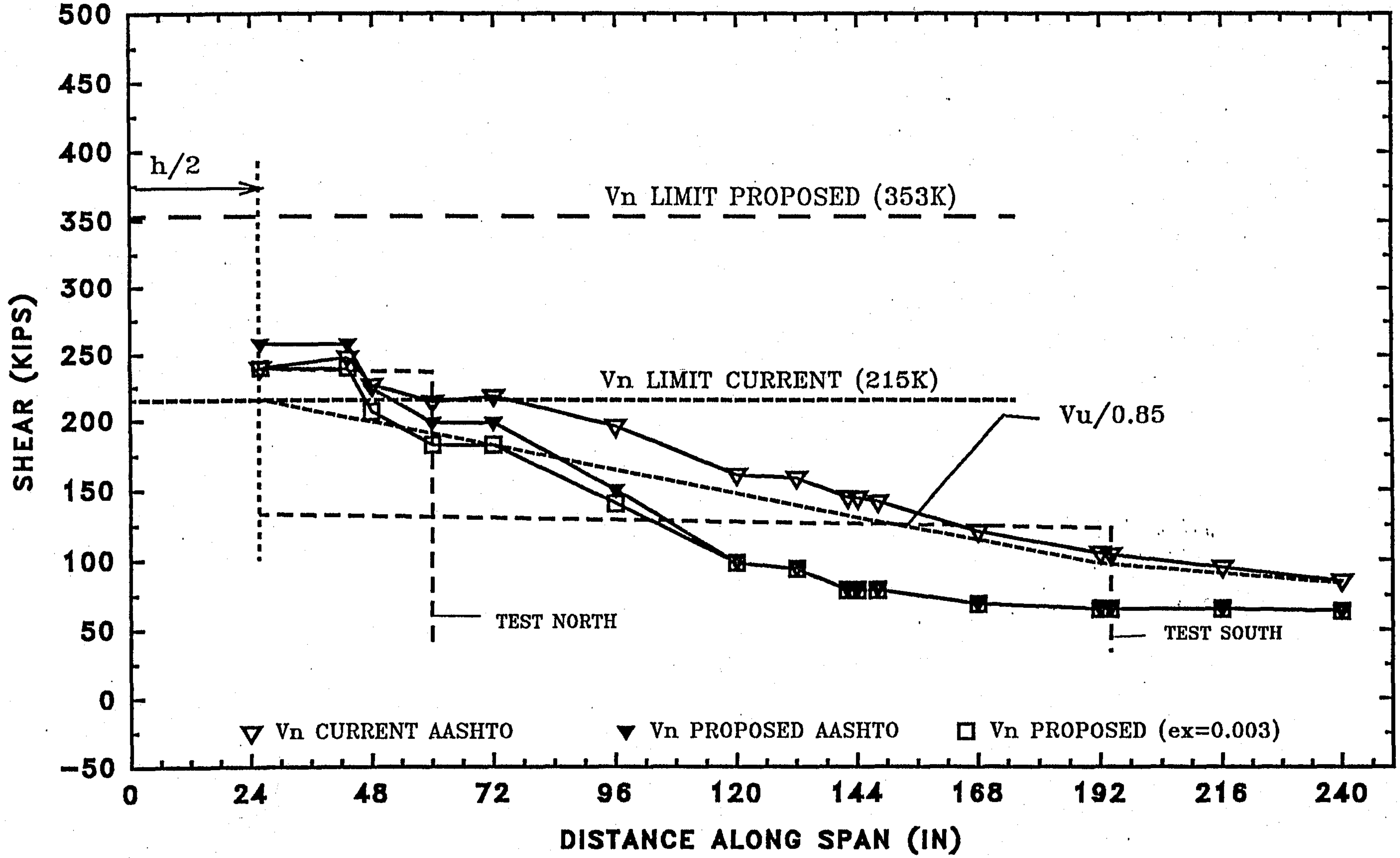


FIGURE A4- 26

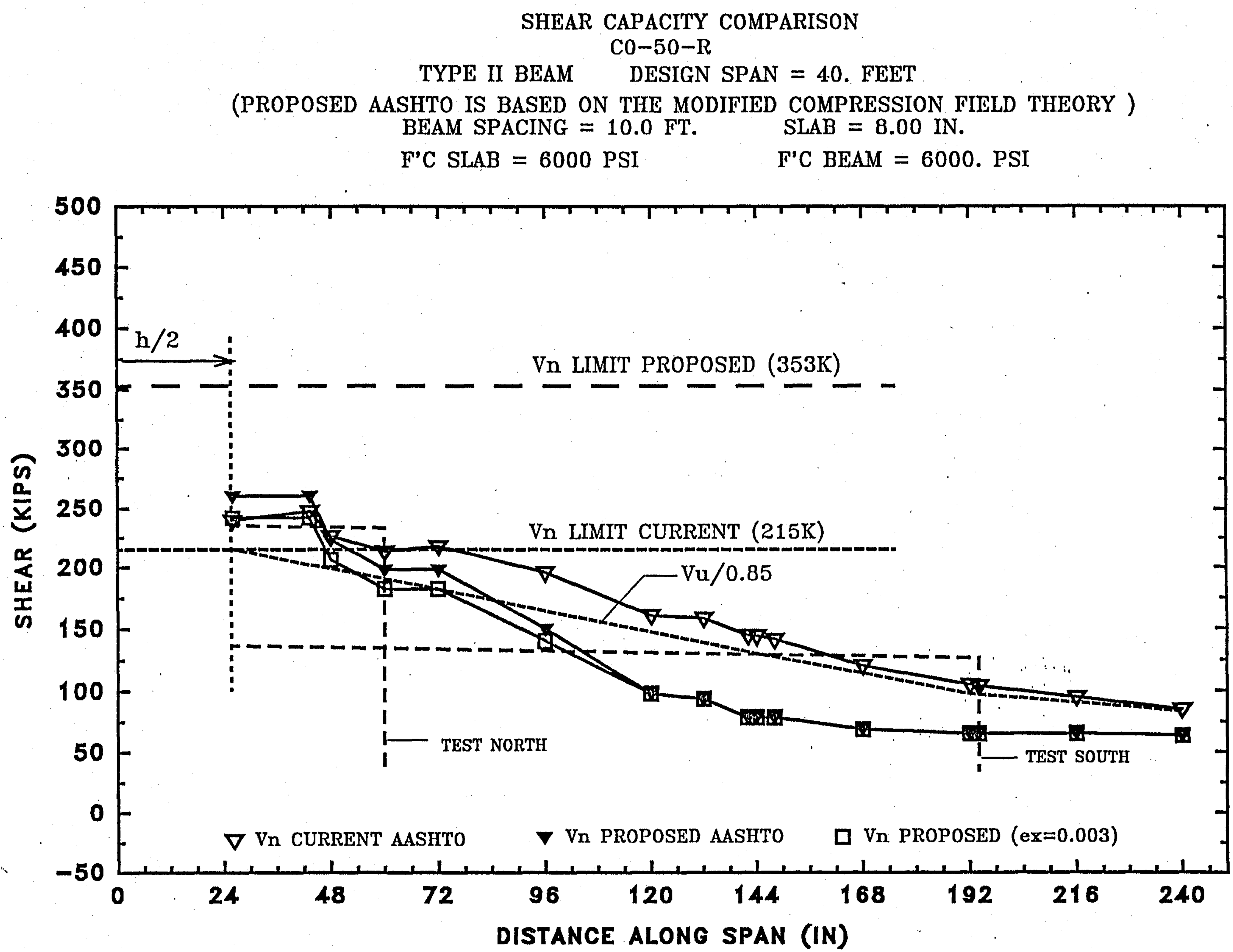

SHEAR CAPACITY COMPARISON
C0-50-R
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI
SHEAR (KIPS)
DISTANCE ALONG SPAN (IN)
h/2
Vn LIMIT PROPOSED (353K)
Vn LIMIT CURRENT (215K)
Vu/0.85
TEST NORTH
TEST SOUTH
Vn CURRENT AASHTO
Vn PROPOSED AASHTO
Vn PROPOSED (ex=0.003)
500
450
400
350
300
250
200
150
100
50
0
-50
0
24
48
72
96
120
144
168
192
216
240

FIGURE A4-27

SHEAR CAPACITY COMPARISON
C1-00-R
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

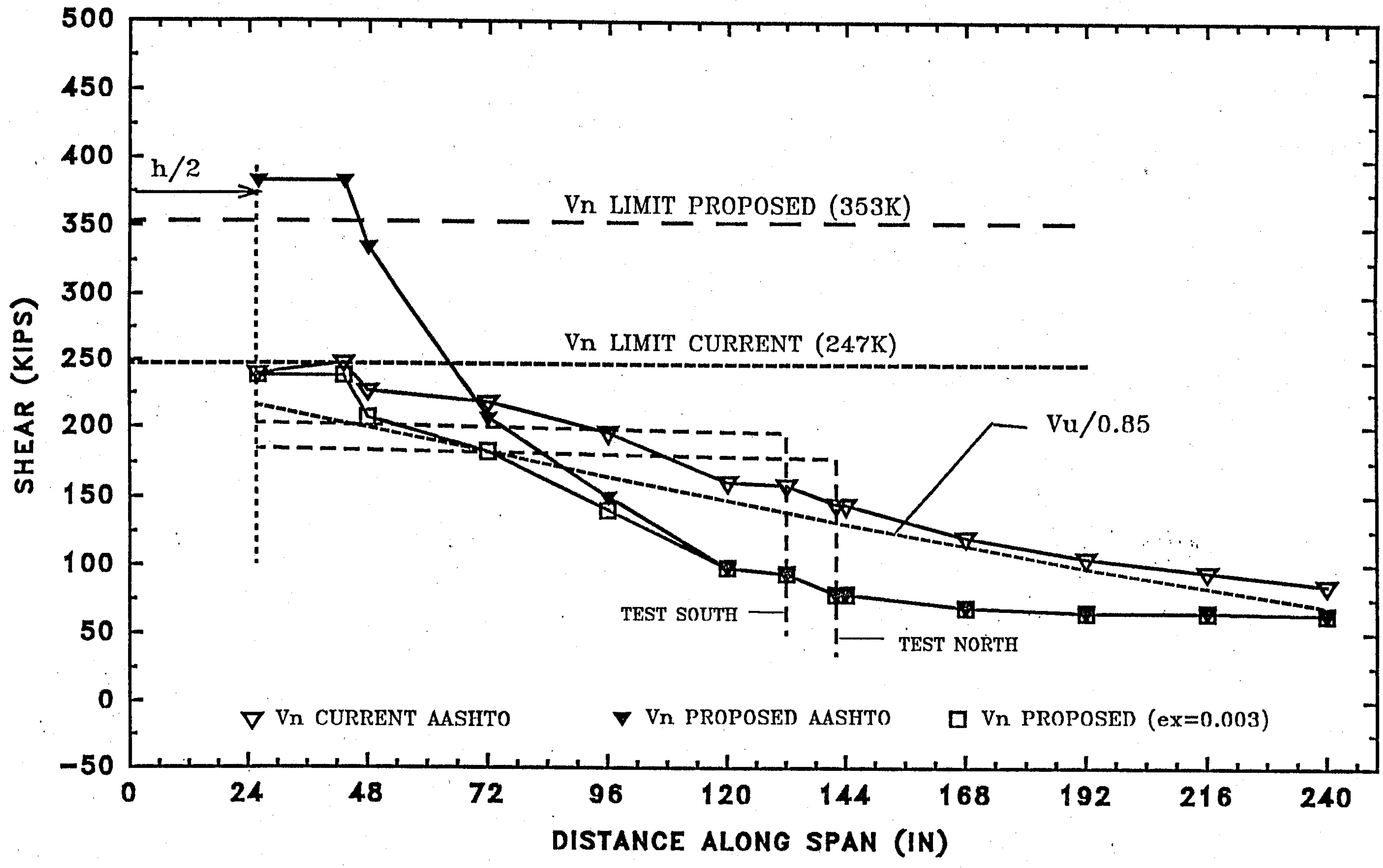


FIGURE A4- 28

SHEAR CAPACITY COMPARISON
C1-00-3R/2
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

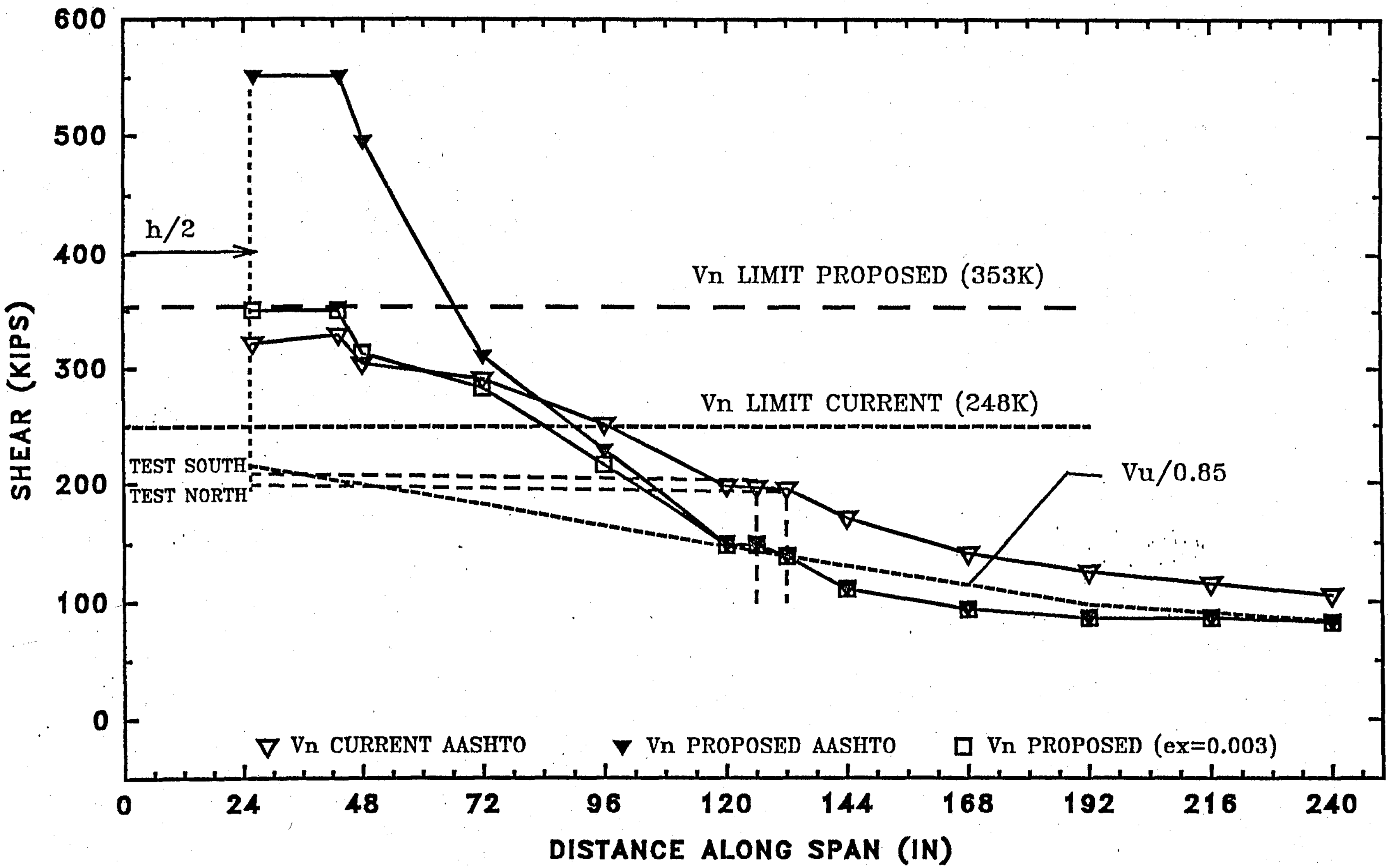


FIGURE A4-29

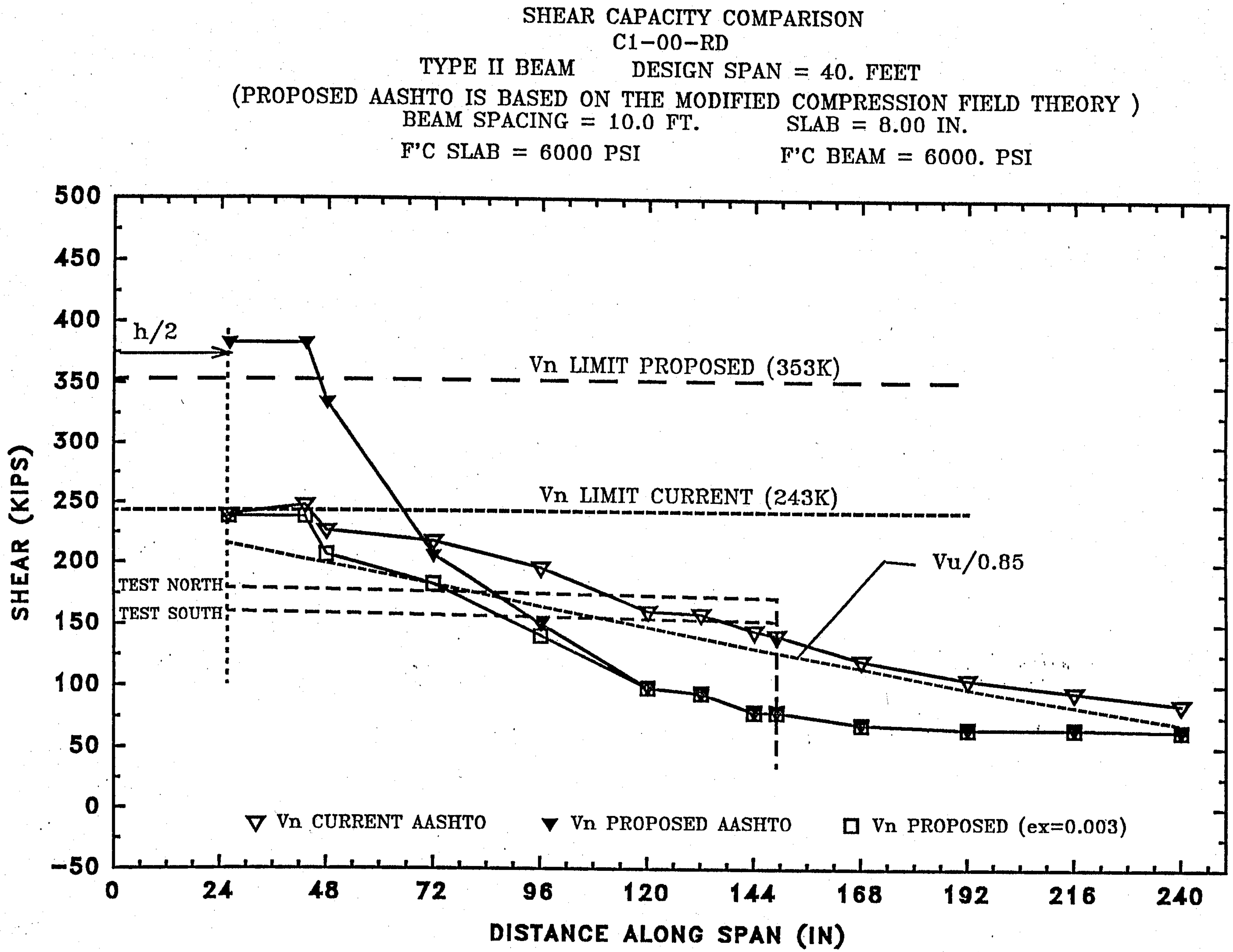

SHEAR CAPACITY COMPARISON
C1-00-RD
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI
h/2
Vn LIMIT PROPOSED (353K)
Vn LIMIT CURRENT (243K)
Vu/0.85
TEST NORTH
TEST SOUTH
Vn CURRENT AASHTO
Vn PROPOSED AASHTO
Vn PROPOSED (ex=0.003)
SHEAR (KIPS)
500
450
400
350
300
250
200
150
100
50
0
-50
0
24
48
72
96
120
144
168
192
216
240
DISTANCE ALONG SPAN (IN)

FIGURE A4-30

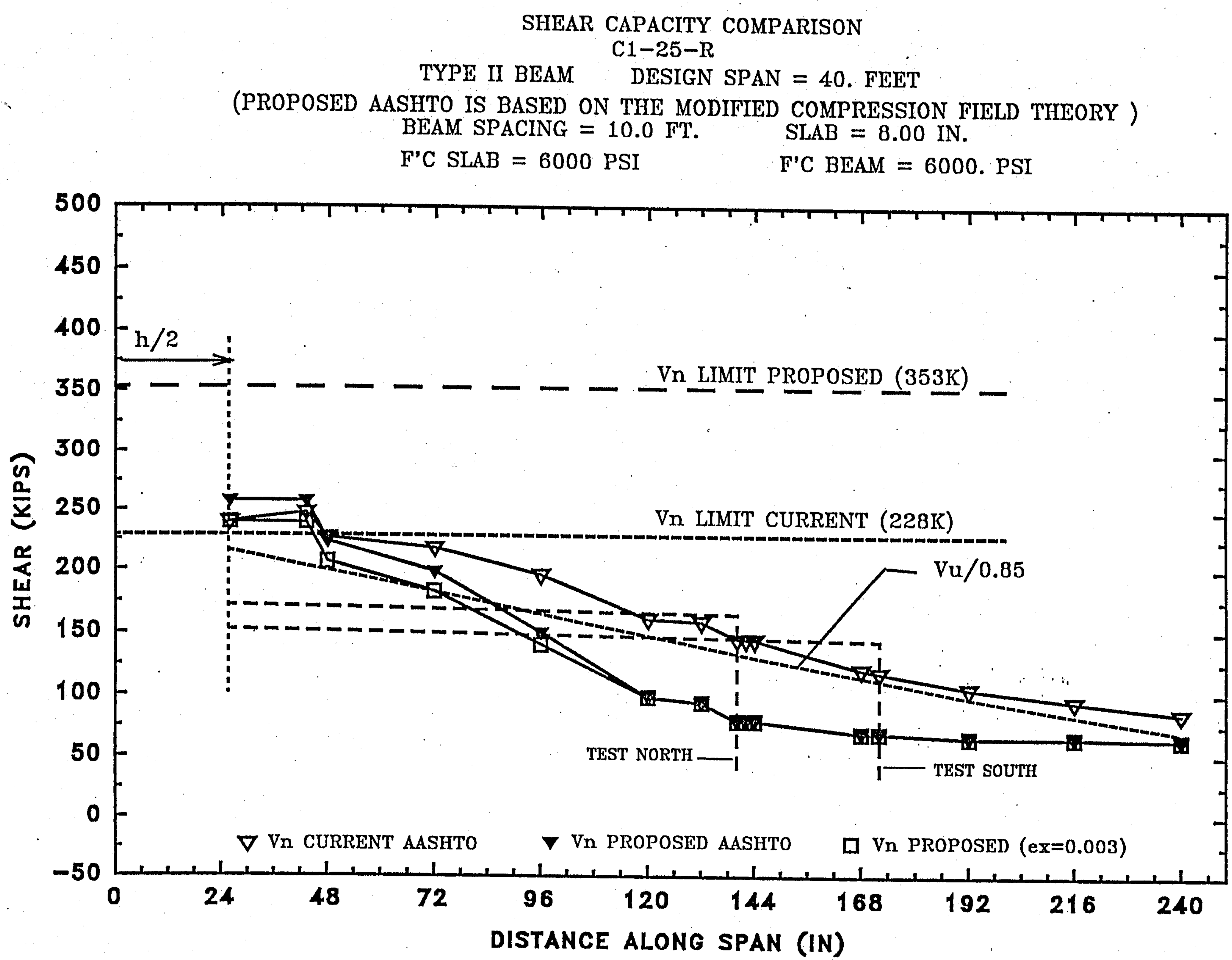

SHEAR CAPACITY COMPARISON
C1-25-R
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI
h/2
Vn LIMIT PROPOSED (353K)
Vn LIMIT CURRENT (228K)
Vu/0.85
TEST NORTH
TEST SOUTH
Vn CURRENT AASHTO
Vn PROPOSED AASHTO
Vn PROPOSED (ex=0.003)
SHEAR (KIPS)
DISTANCE ALONG SPAN (IN)
500
450
400
350
300
250
200
150
100
50
0
-50
0
24
48
72
96
120
144
168
192
216
240

FIGURE A4- 31

SHEAR CAPACITY COMPARISON
C1-50-R
TYPE II BEAM   DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT.   SLAB = 8.00 IN.
F'C SLAB = 6000 PSI   F'C BEAM = 6000. PSI

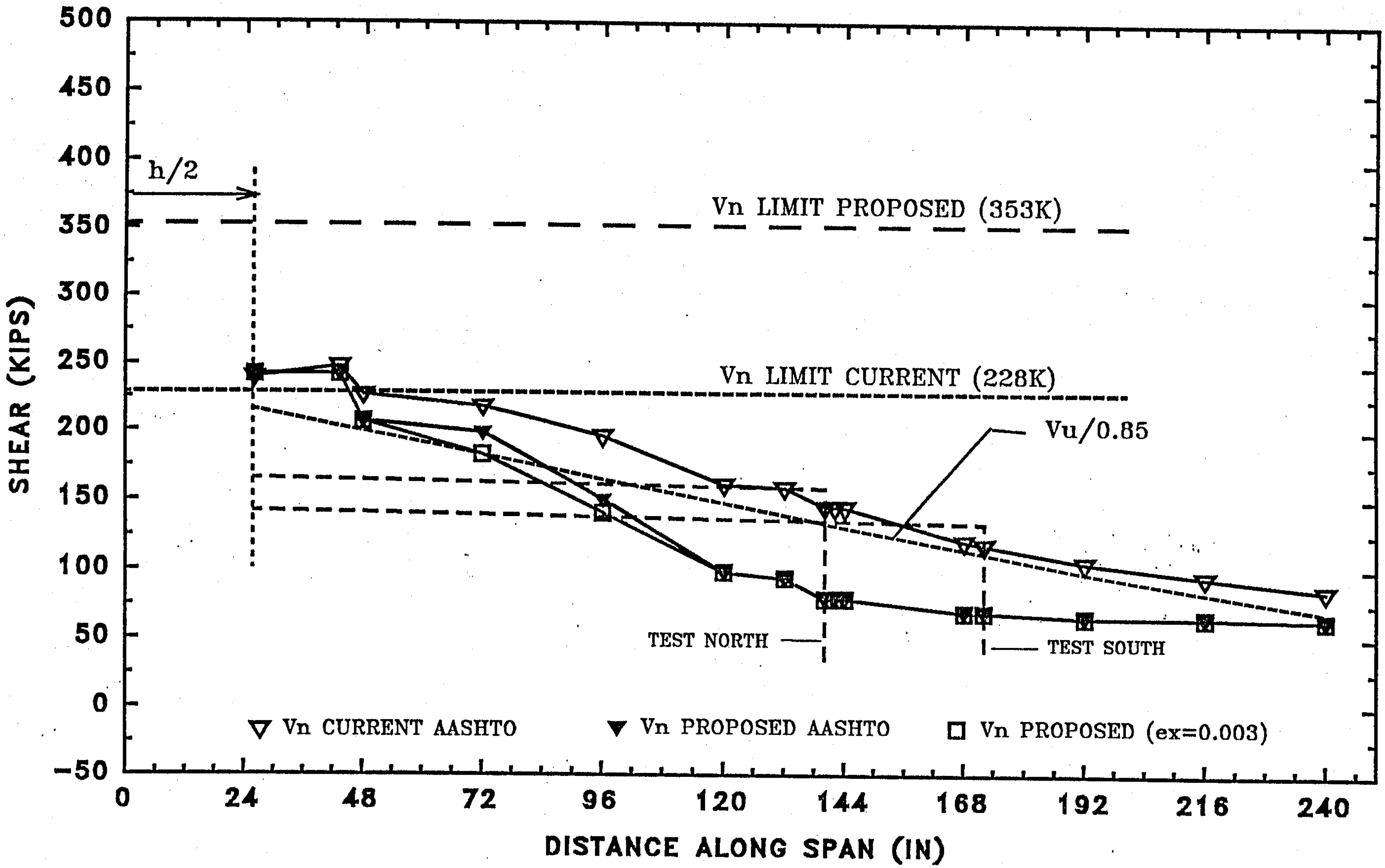


FIGURE A4-32

SHEAR CAPACITY COMPARISON
A1-00-M
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

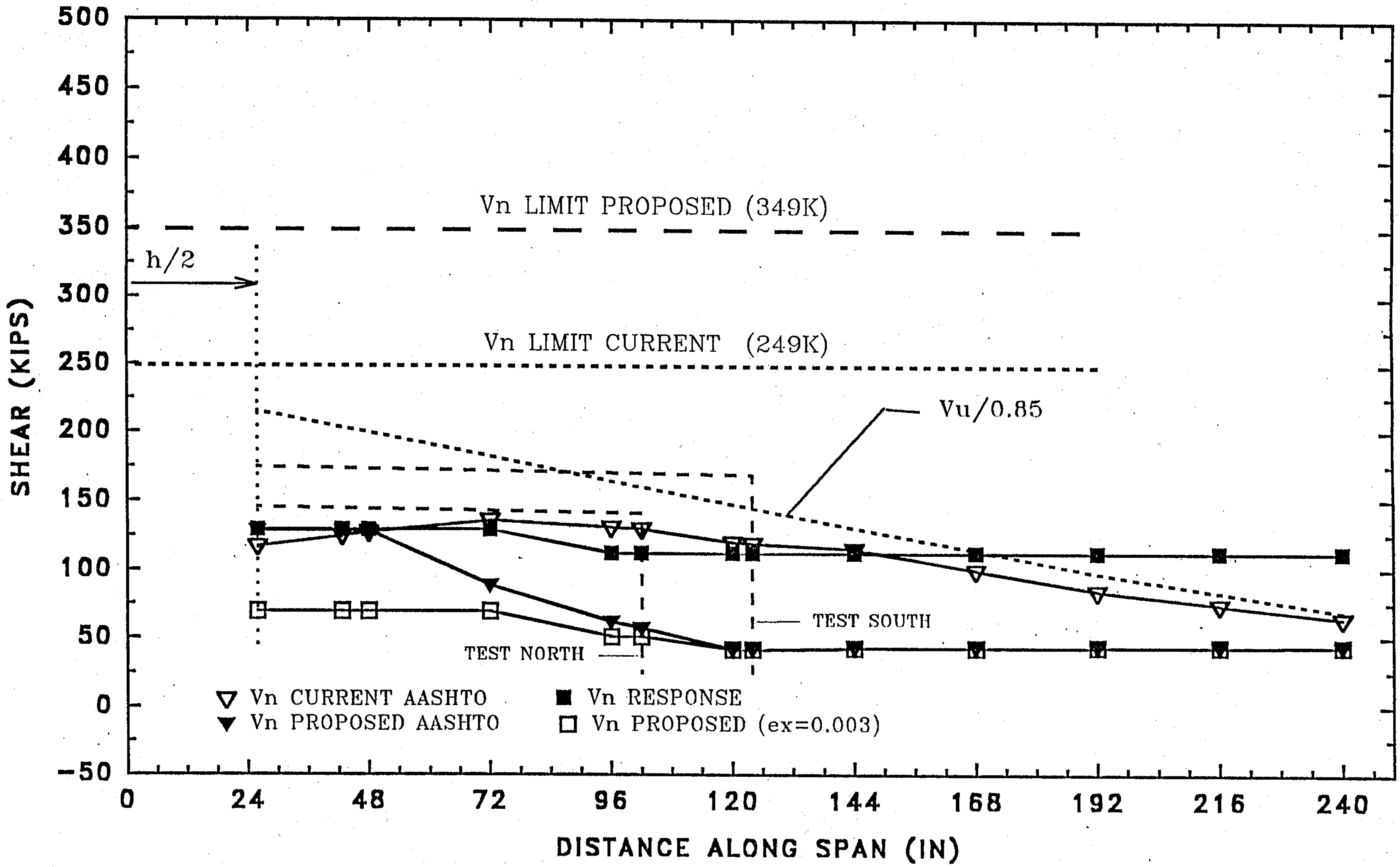


FIGURE A4-33

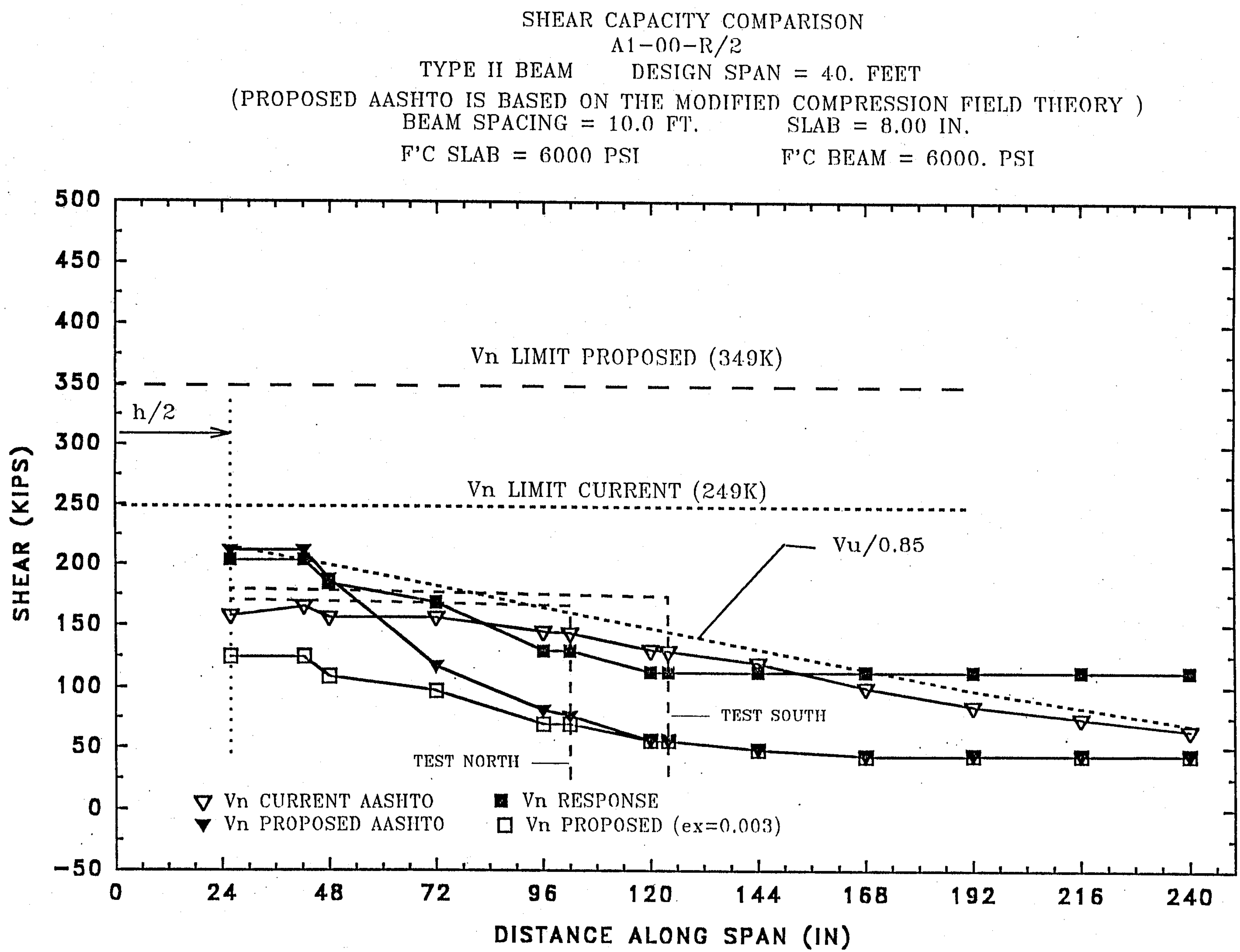

SHEAR CAPACITY COMPARISON
A1-00-R/2
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI
Vn LIMIT PROPOSED (349K)
Vn LIMIT CURRENT (249K)
h/2
Vu/0.85
TEST SOUTH
TEST NORTH
Vn CURRENT AASHTO
Vn PROPOSED AASHTO
Vn RESPONSE
Vn PROPOSED (ex=0.003)
SHEAR (KIPS)
DISTANCE ALONG SPAN (IN)
500
450
400
350
300
250
200
150
100
50
0
-50
0
24
48
72
96
120
144
168
192
216
240

FIGURE A4-34

SHEAR CAPACITY COMPARISON
A1-00-R
TYPE II BEAM DESIGN SPAN = 40. FEET
(PROPOSED AASHTO IS BASED ON THE MODIFIED COMPRESSION FIELD THEORY)
BEAM SPACING = 10.0 FT. SLAB = 8.00 IN.
F'C SLAB = 6000 PSI F'C BEAM = 6000. PSI

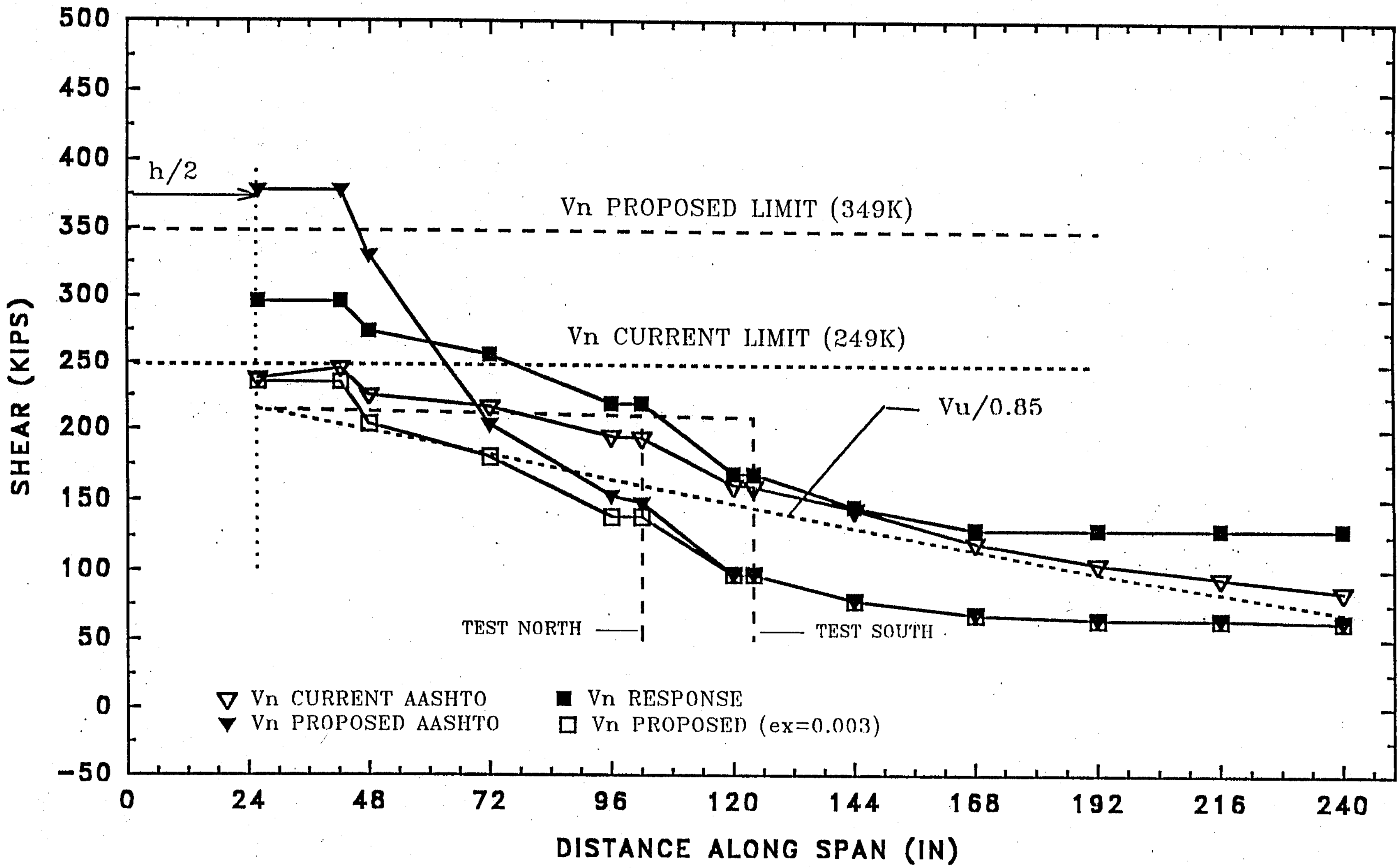
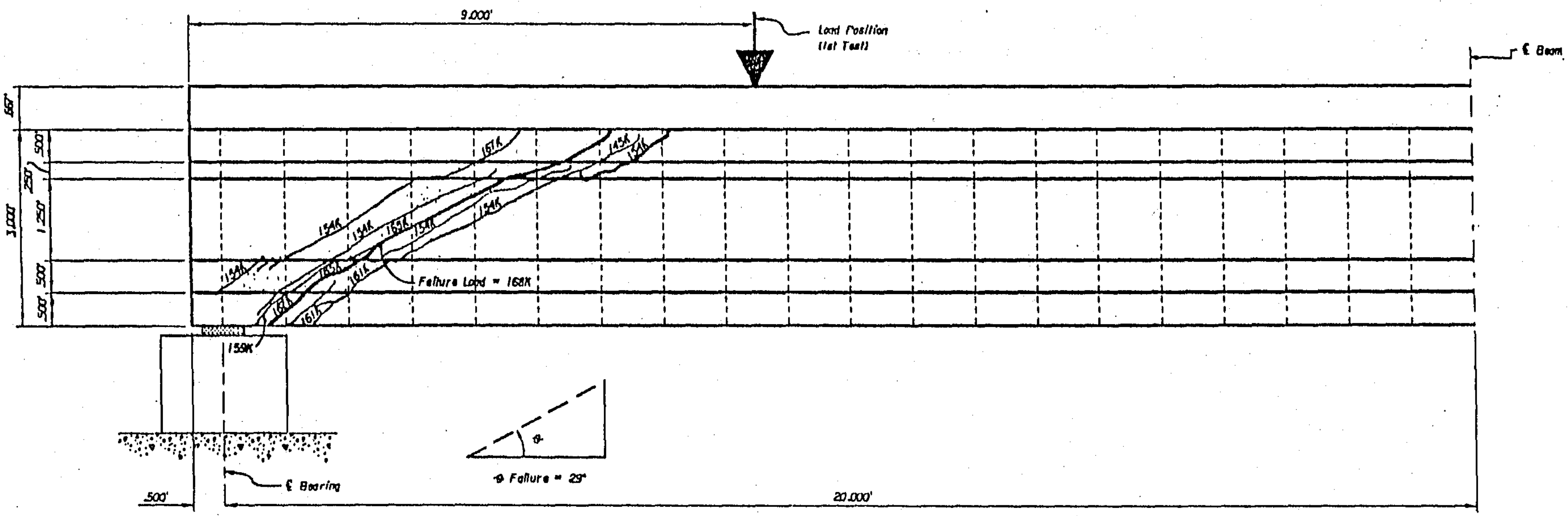


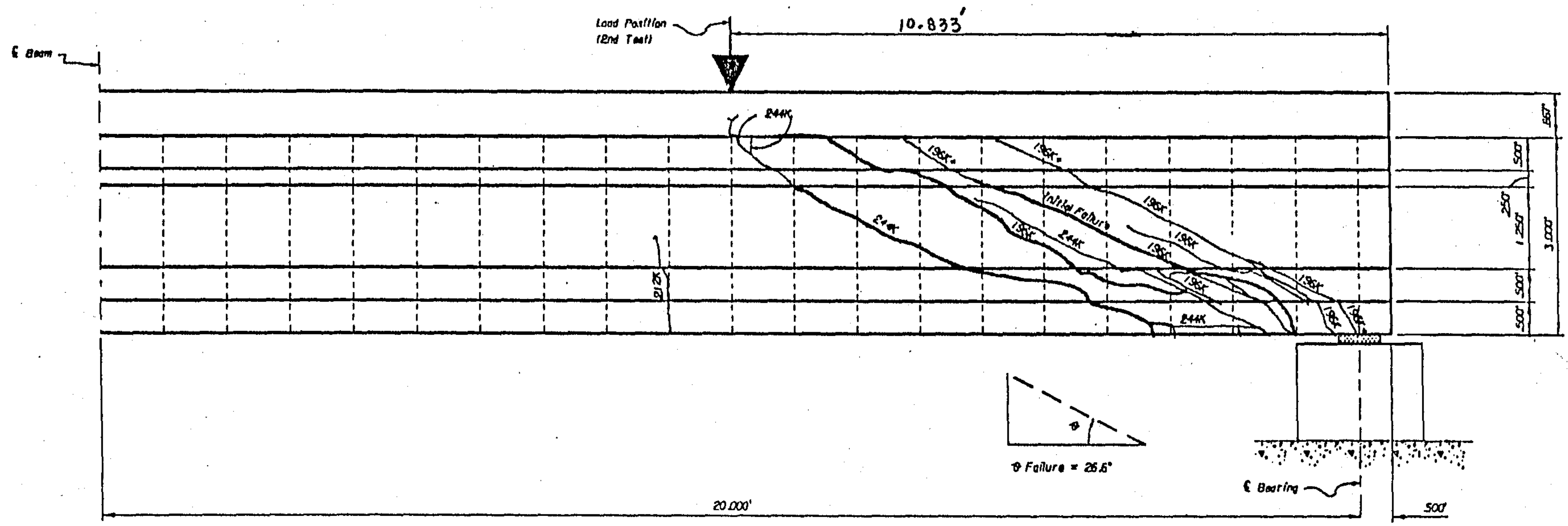
FIGURE A4-35

Appendix A5

Girder Crack Patterns

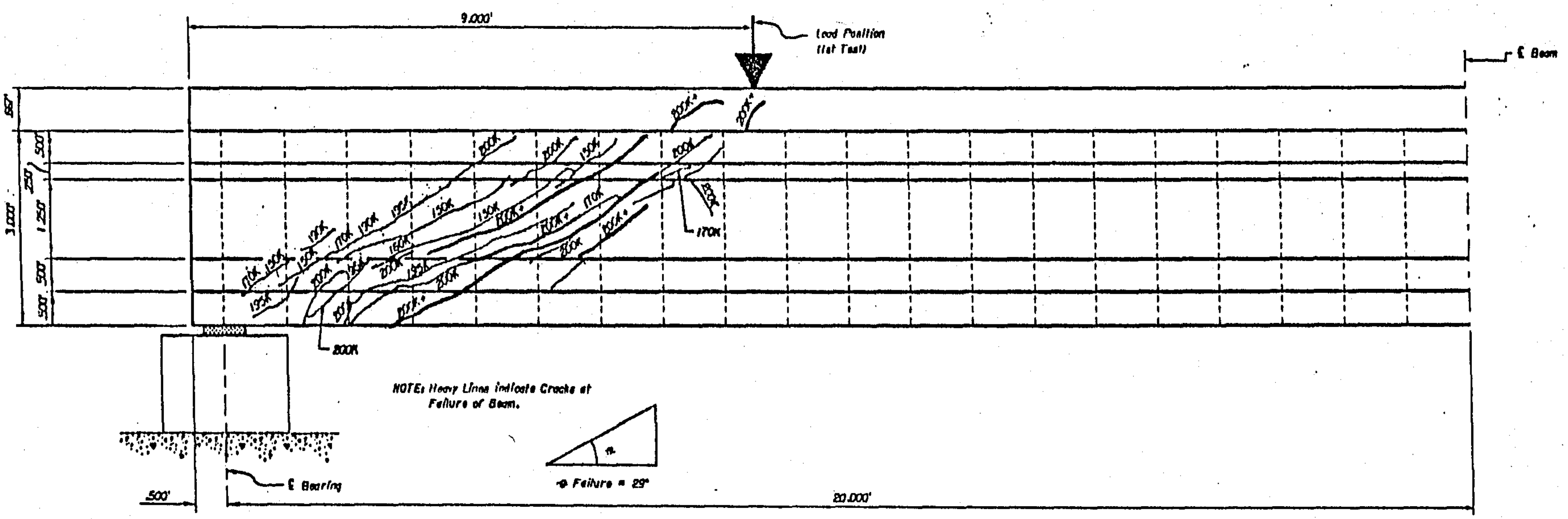


BEAM CRACK CHART
(SPAN 40.00')
A1-00-M (North)

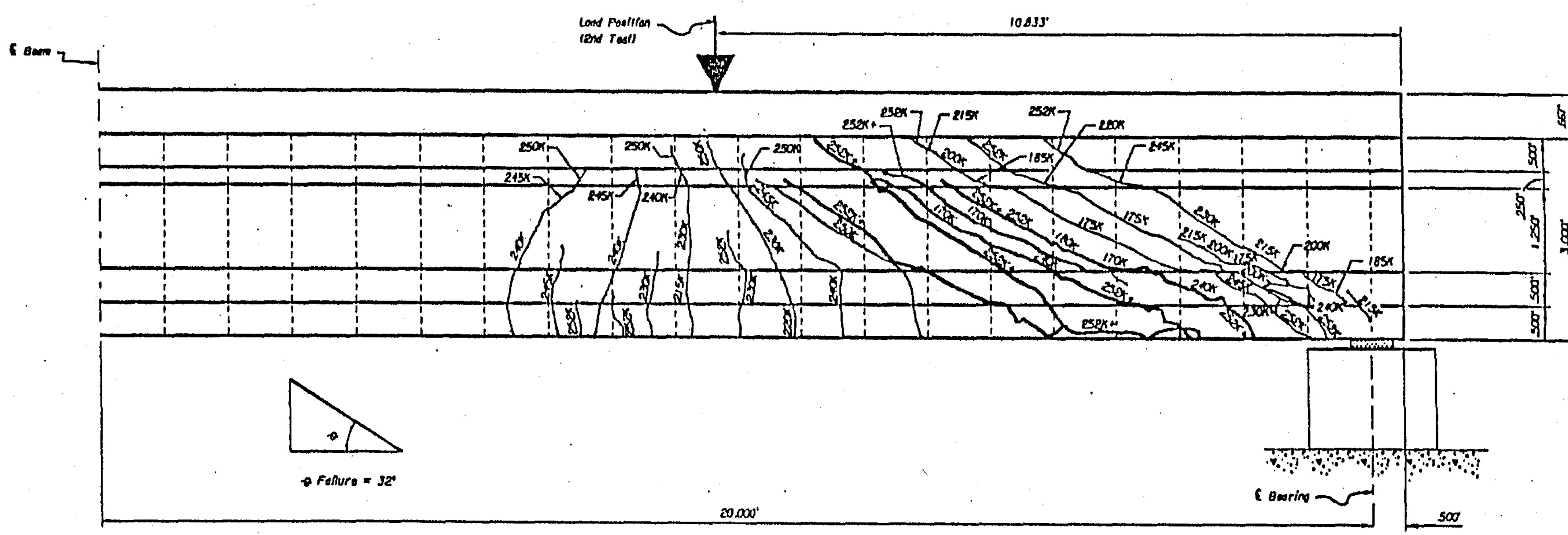


BEAM CRACK CHART
(SPAN 31.50')
A1-00-M (South)

FIGURE A5-1

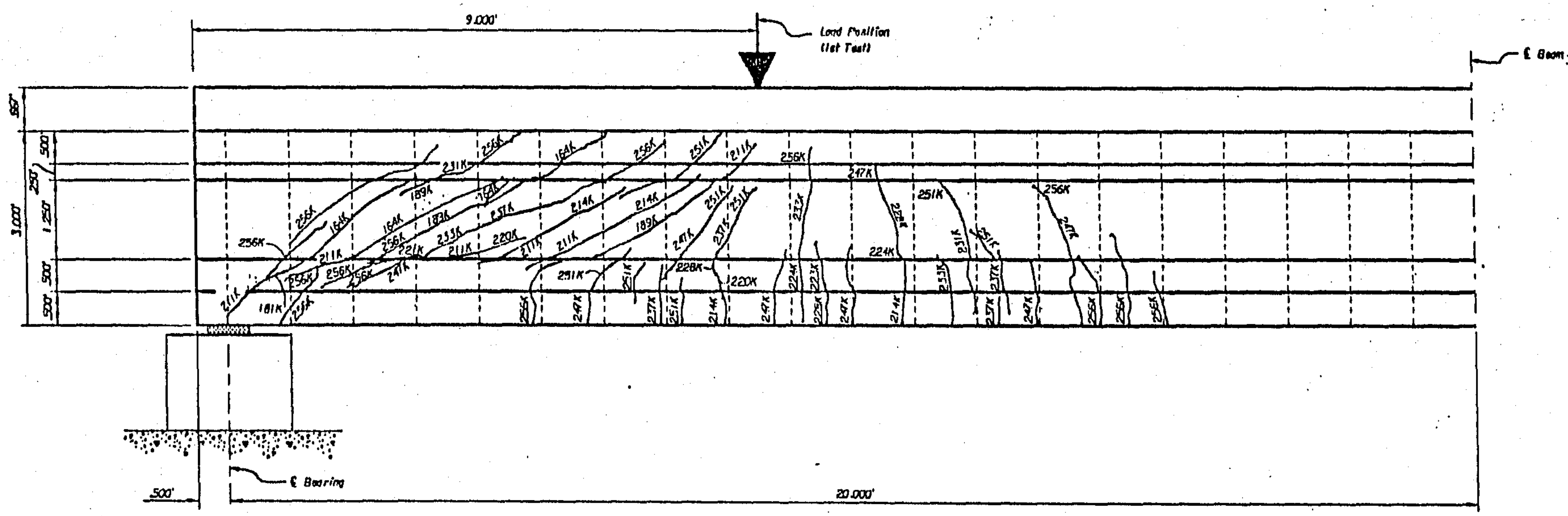


BEAM CRACK CHART
(SPAN 10.00')
A1-00-R/2 (North)



BEAM CRACK CHART
(SPAN 31.50')
A1-00-R/2 (South)

FIGURE A5-2



BEAM CRACK CHART
(SPAN 40.00')
A1-00- R North(10-15-90)

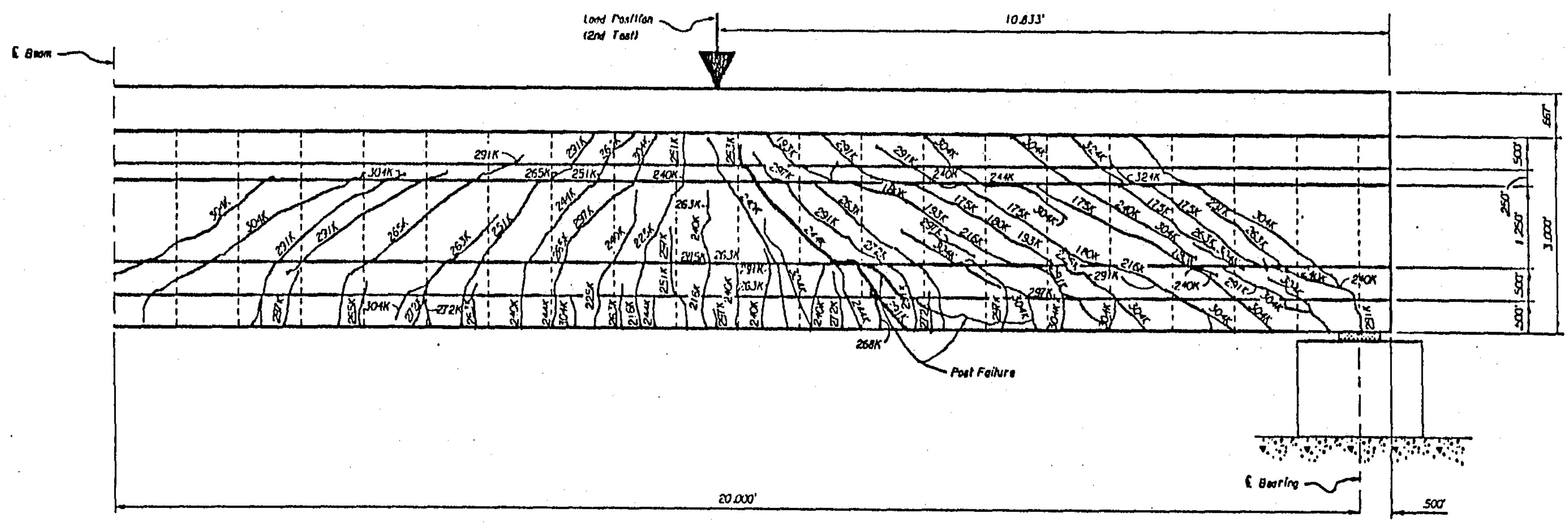
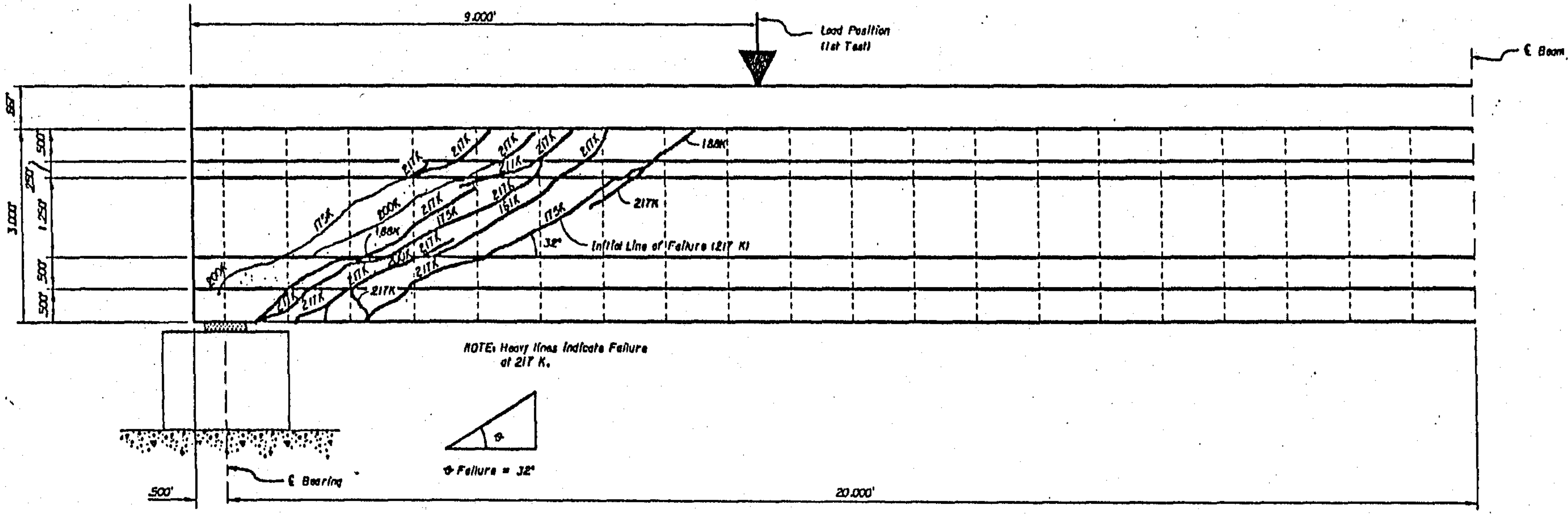


FIGURE A5-3

BEAM CRACK CHART
(SPAN 31.50')
A1-00-R South(10-16-90)



BEAM CRACK CHART
(SPAN 40.00')
A1-00-RD (North)

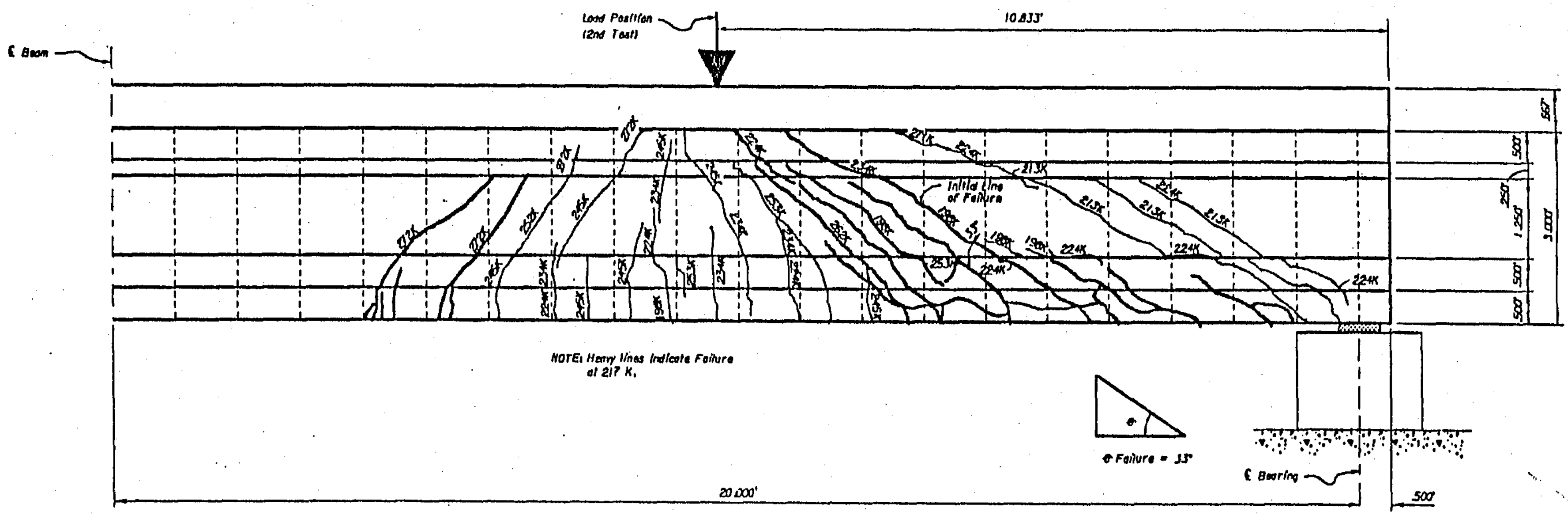
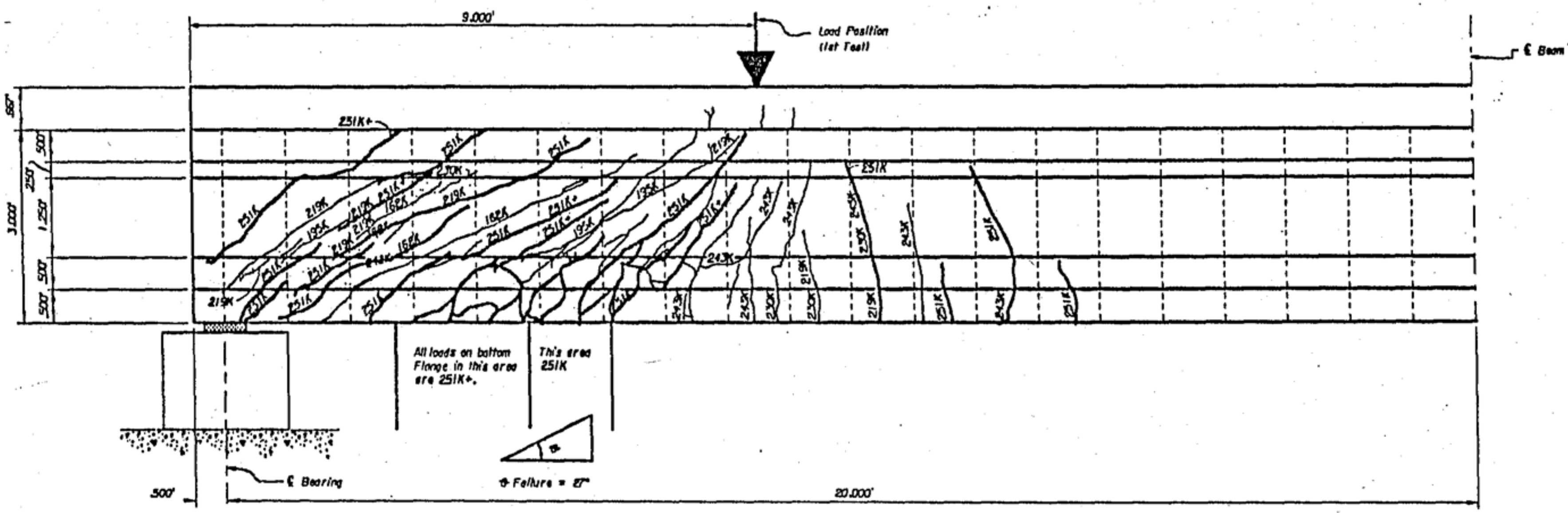
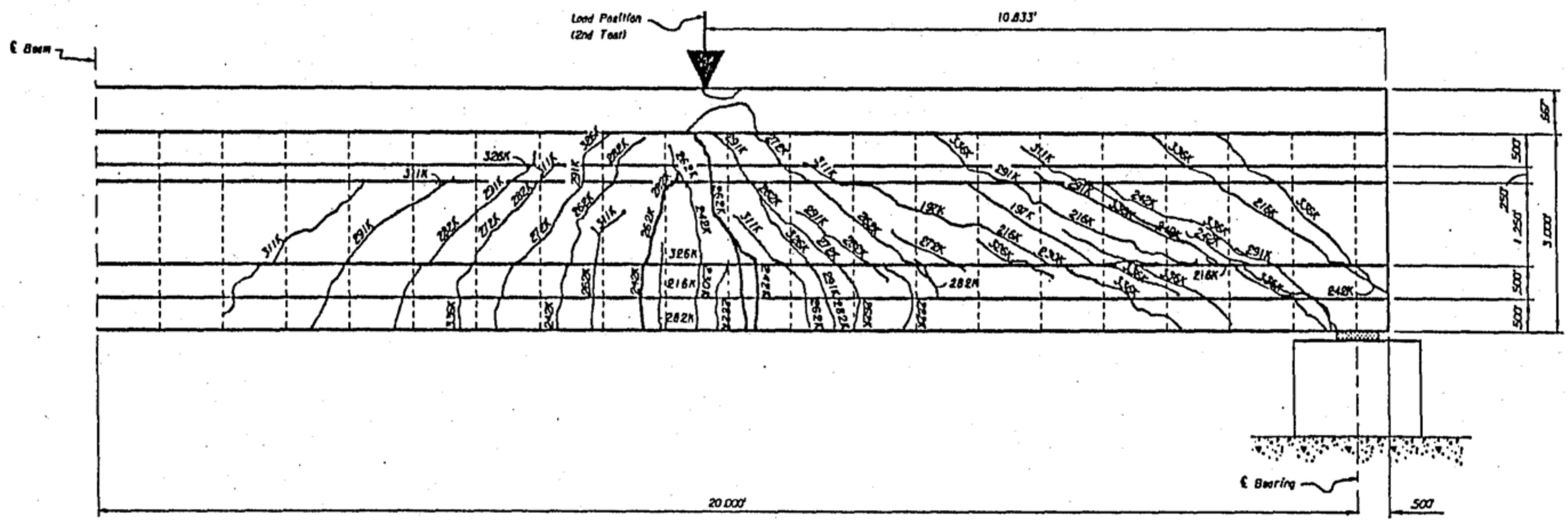


FIGURE A5-4

BEAM CRACK CHART
(SPAN 31.50')
A1-00-RD (South)

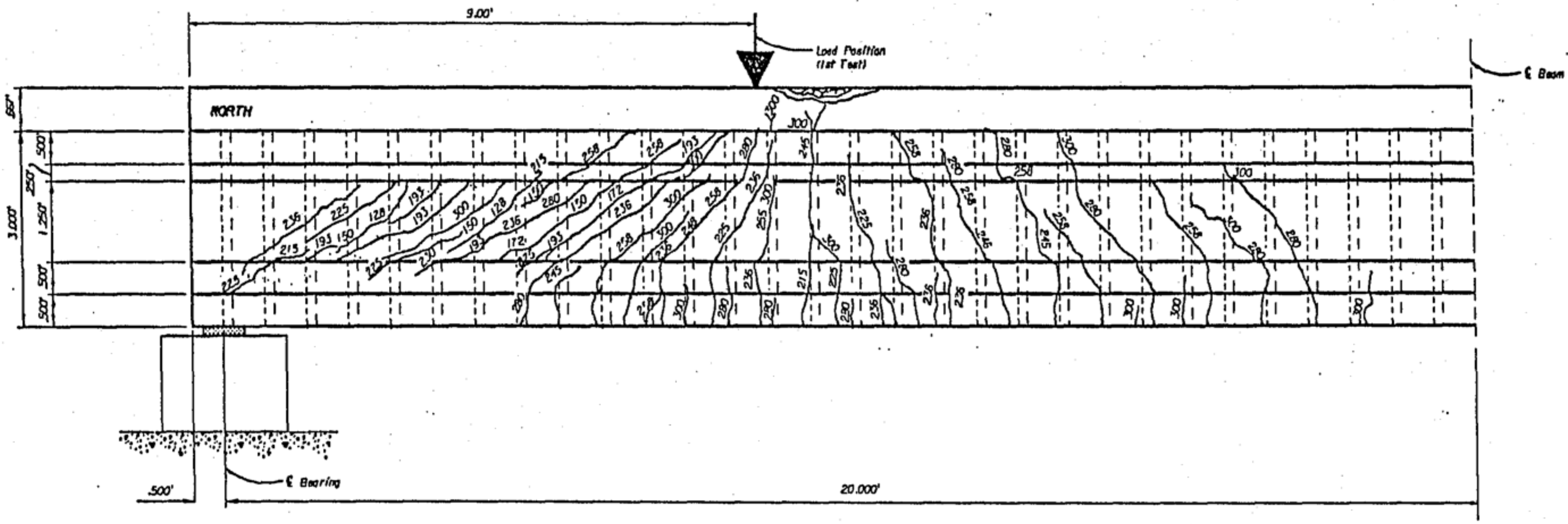


BEAM CRACK CHART
(SPAN 40.00')
A1-00-3R/2 (North)



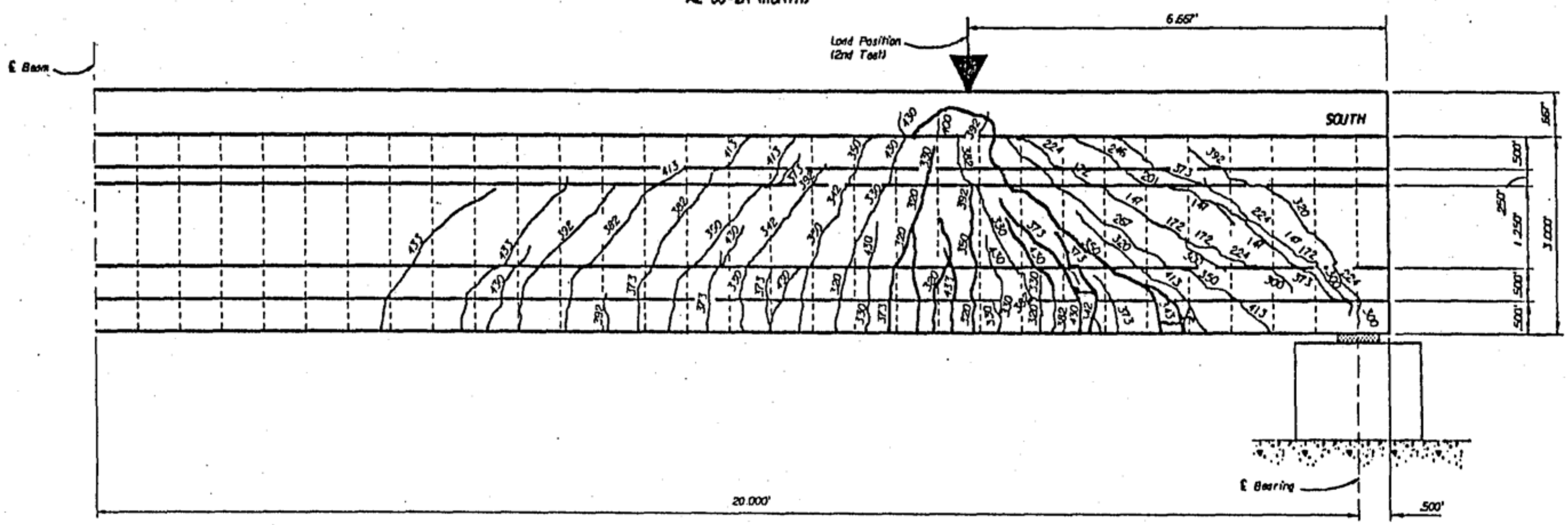
BEAM CRACK CHART
(SPAN 31.50')
A1-00-3R/2 (South)

FIGURE A5-5



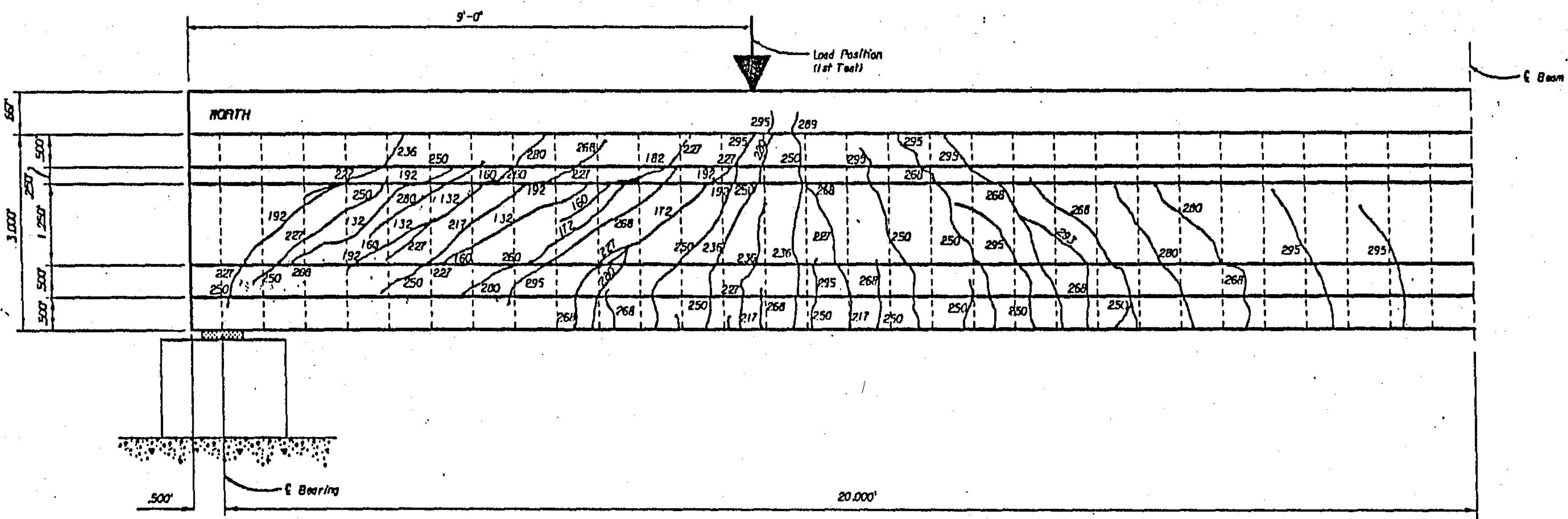
BEAM CRACK CHART
(SPAN 40.00')
A2-00-2R (NORTH)

NOTE: Heavy Lines Indicate Cracks at Failure of Beam.

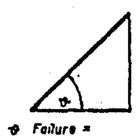


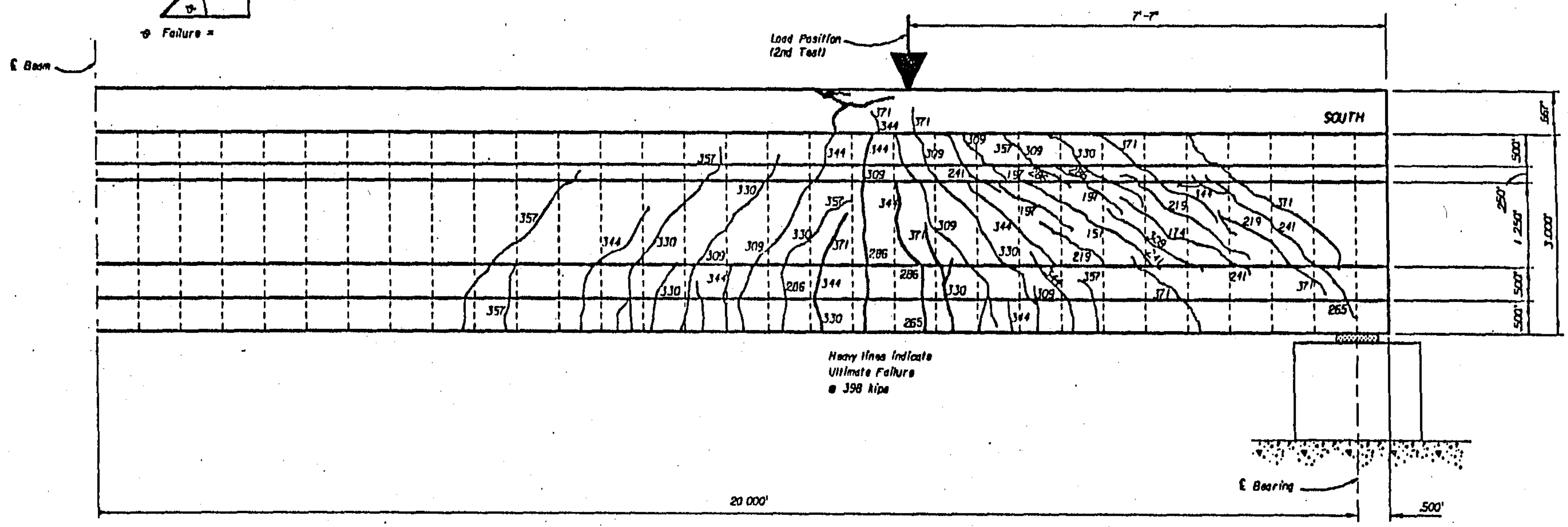
BEAM CRACK CHART
(SPAN 31.5')
A2-00-2R (SOUTH)

FIGURE A5-6



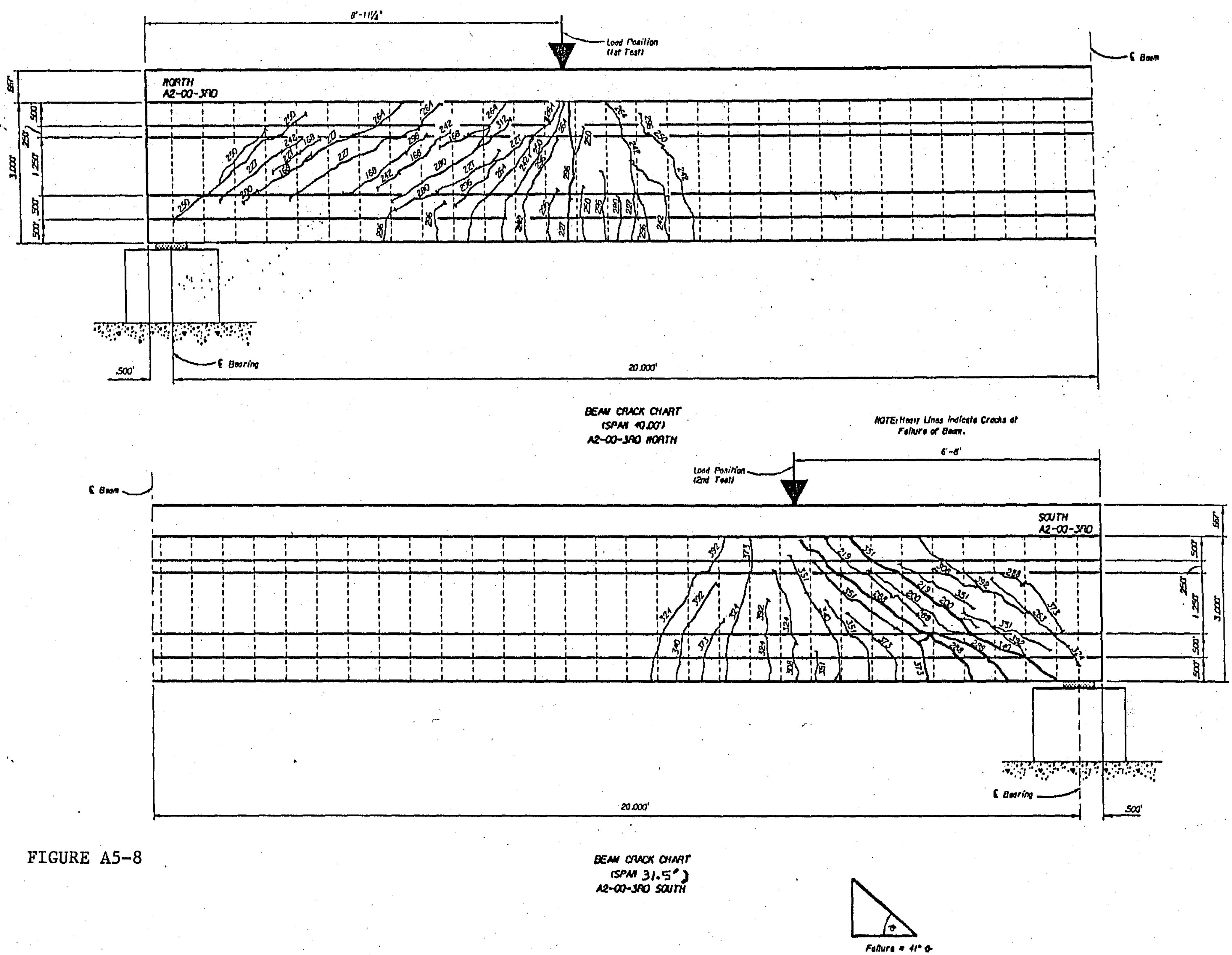
BEAM CRACK CHART
(SPAN 40.00')
A2-00-3R NORTH





BEAM CRACK CHART
(SPAN 31.50')
A2-00-3R SOUTH

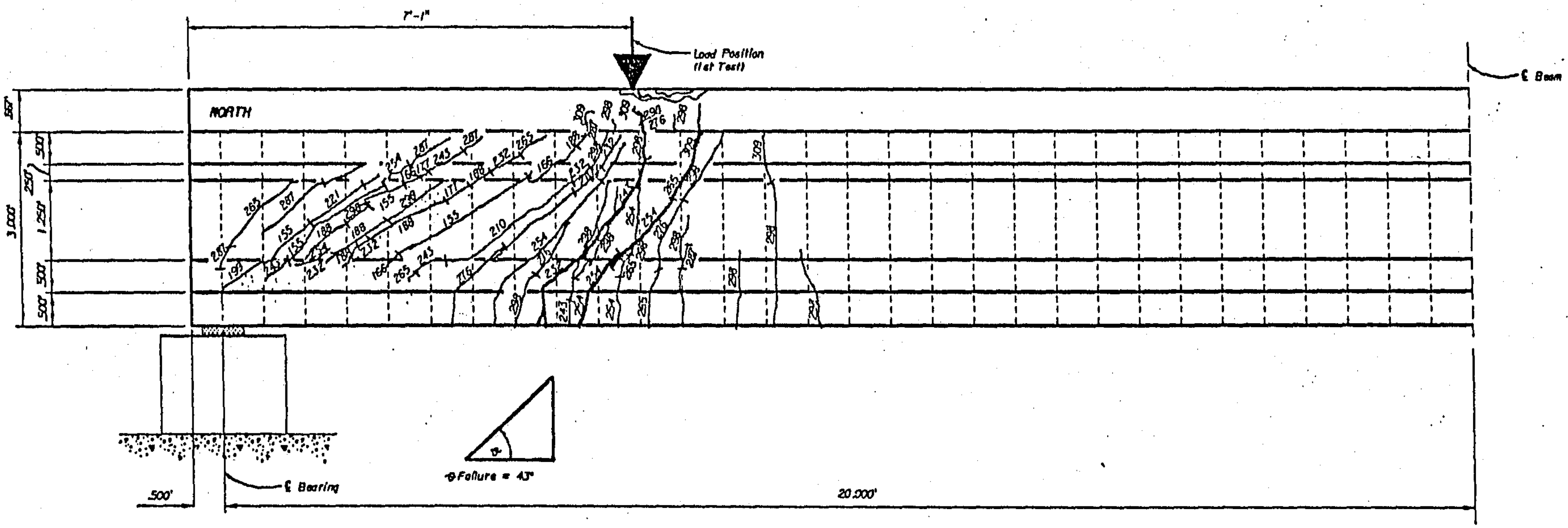
FIGURE A5-7



BEAM CRACK CHART
(SPAN 40.00')
A2-00-3RD NORTH

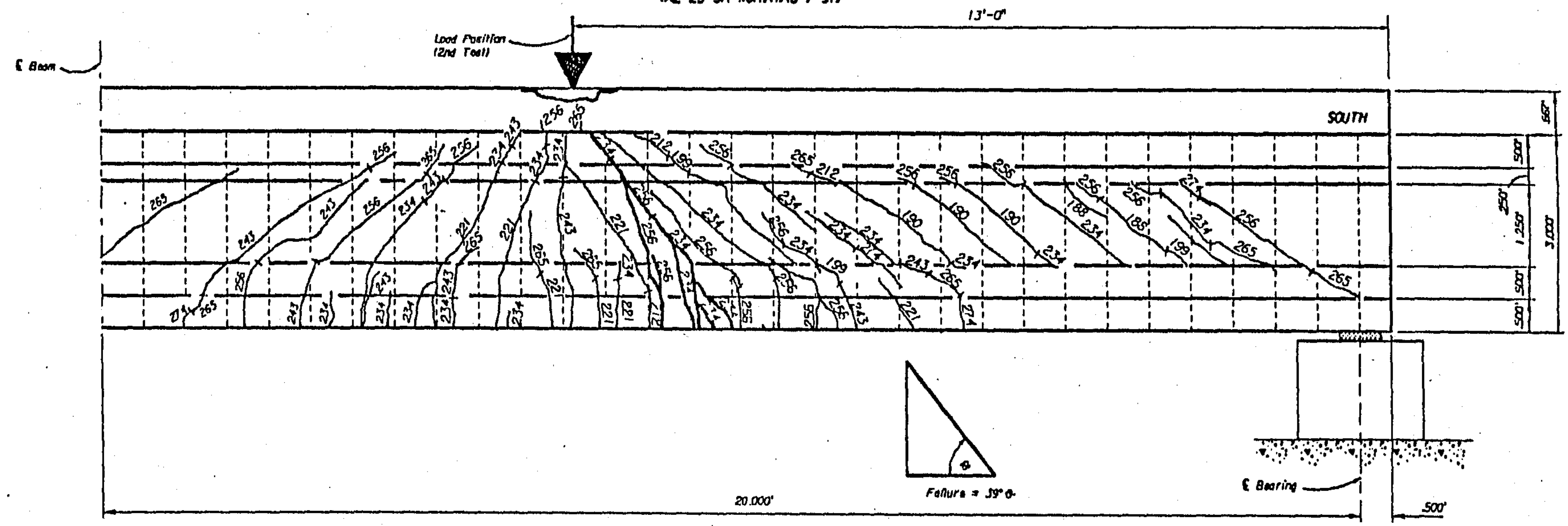
BEAM CRACK CHART
(SPAN 31.5')
A2-00-3RD SOUTH

FIGURE A5-8



BEAM CRACK CHART
(SPAN 40.00')
(A2-25-3R NORTH)(8-7-91)

NOTE: Heavy Lines Indicate Cracks at Failure of Beam.



BEAM CRACK CHART
(SPAN 31.5')
(A2-25-3R SOUTH)(8-8-91)

FIGURE A5-9

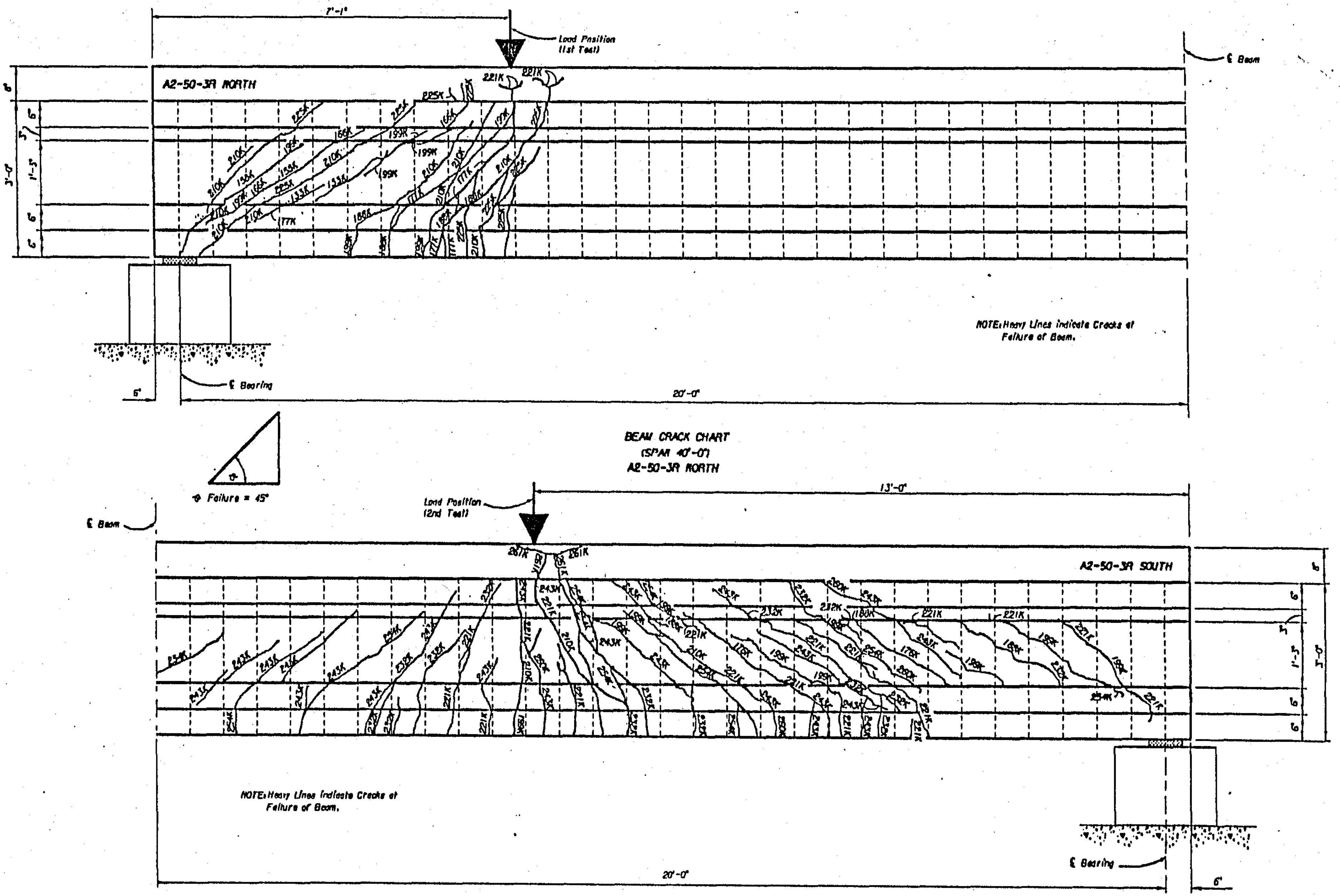
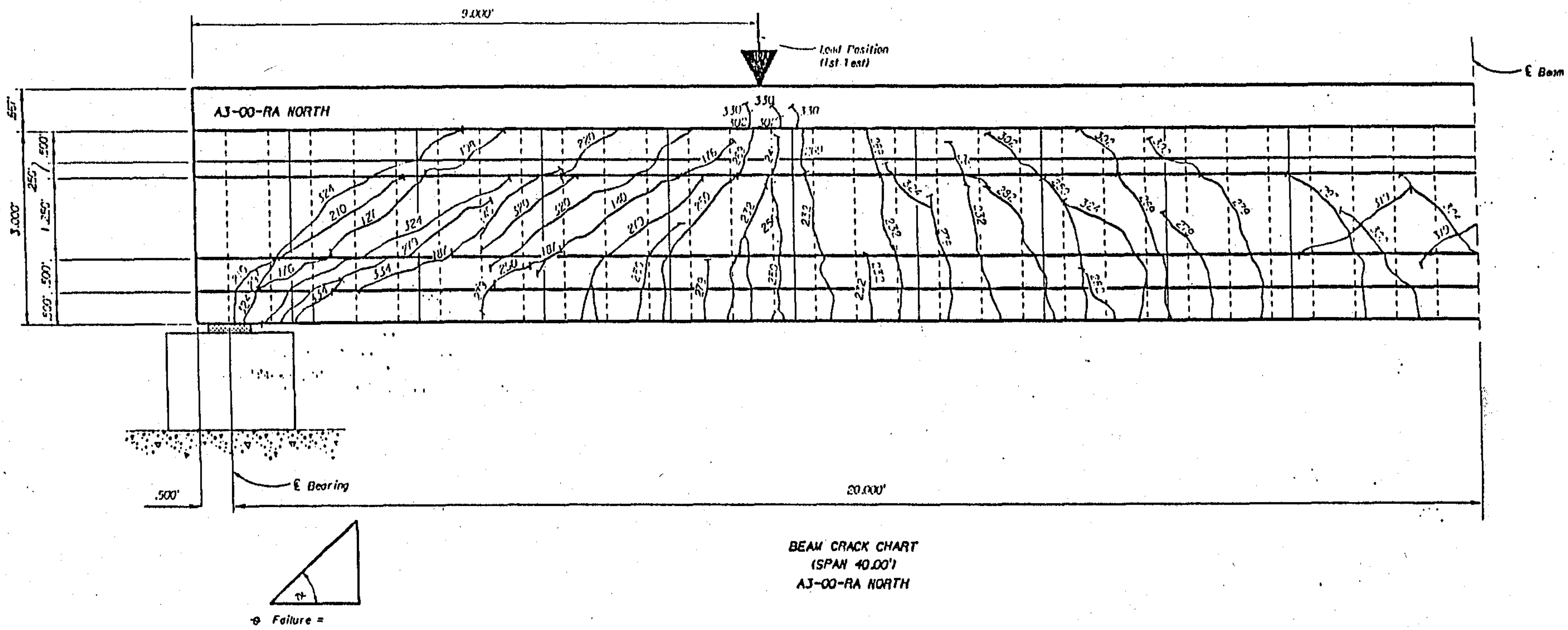


FIGURE A5-10

BEAM CRACK CHART
(SPAN 31'-6")
A2-50-3R SOUTH



BEAM CRACK CHART
(SPAN 40.00')
A3-00-RA NORTH

NOTE: Heavy Lines indicate Cracks at Failure of Beam.

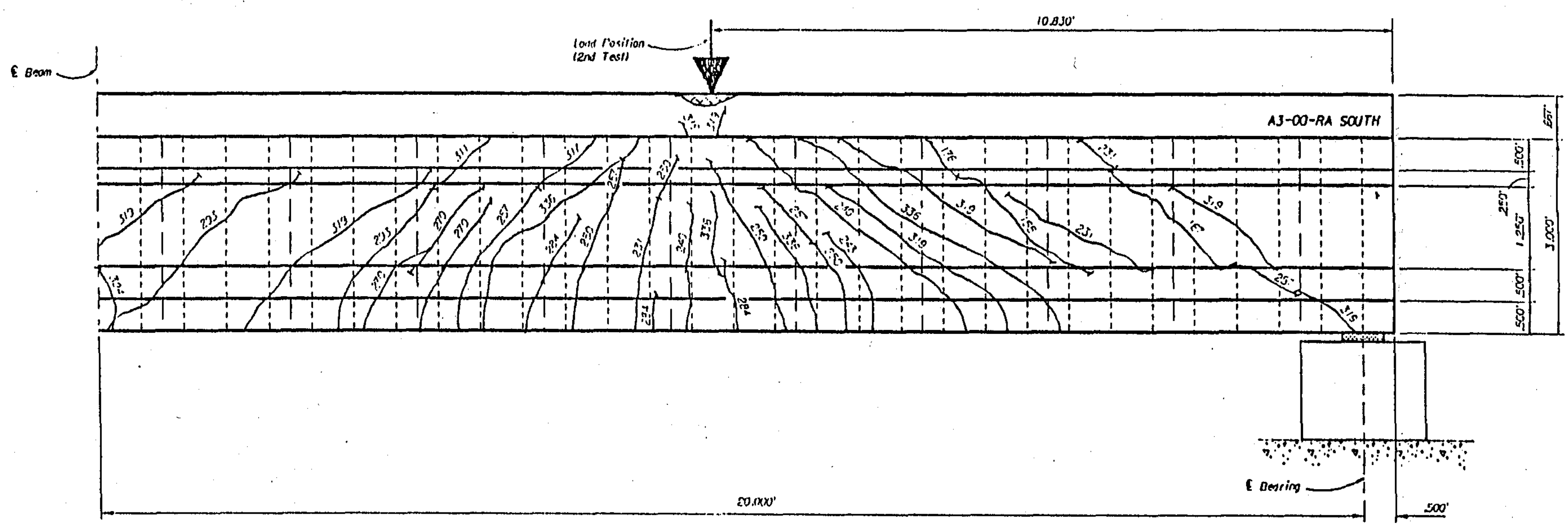
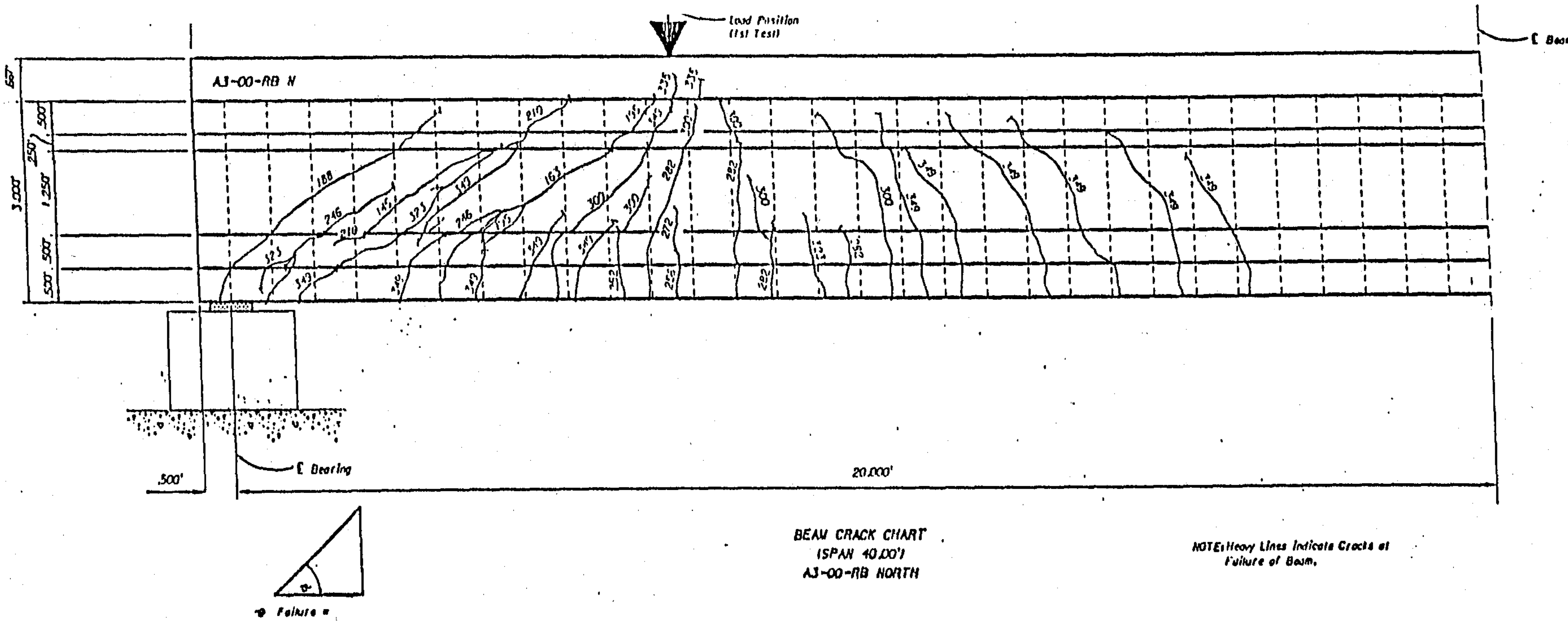


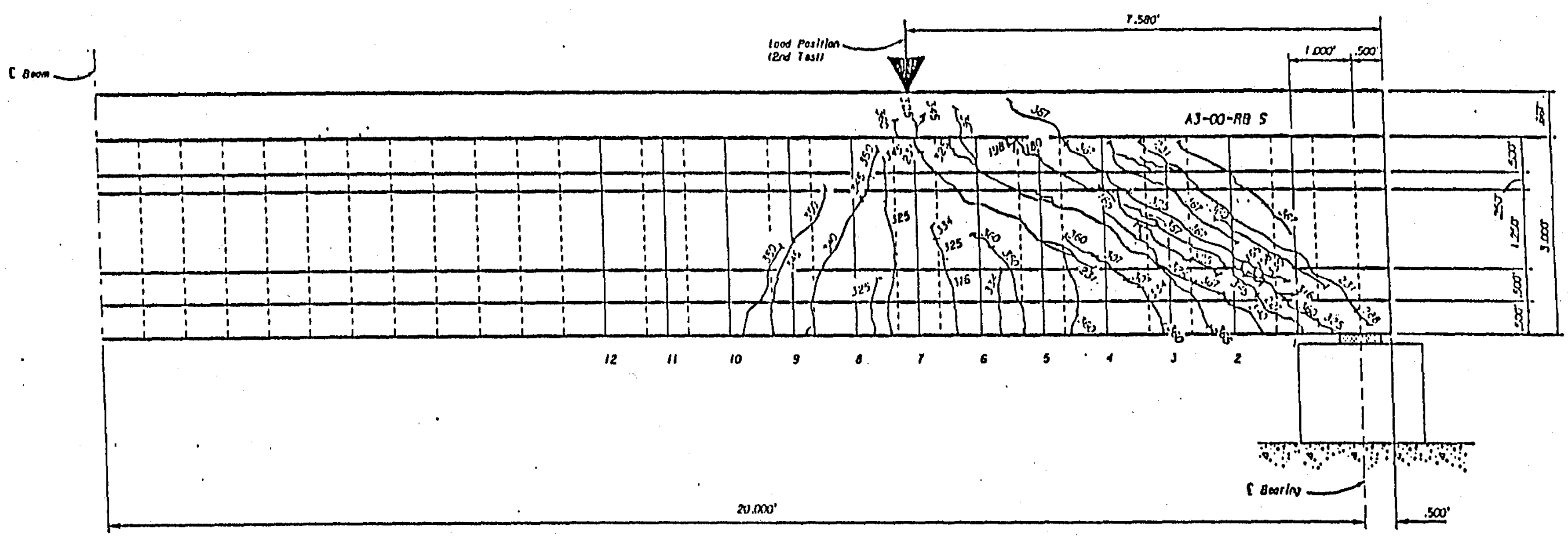
FIGURE A5-11

BEAM CRACK CHART
(SPAN 31.50')
A3-00-RA SOUTH



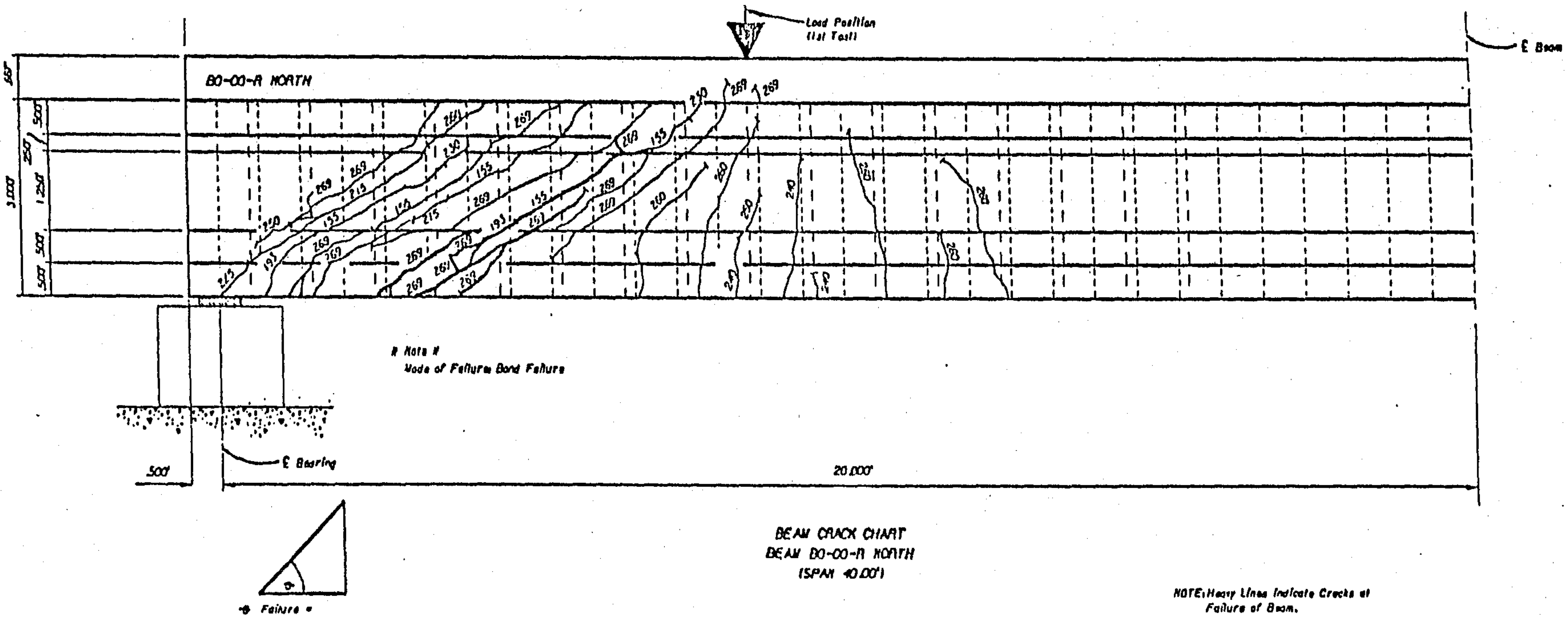
BEAM CRACK CHART
(SPAN 40.00')
A3-00-RB NORTH

NOTE: Heavy Lines Indicate Cracks at Failure of Beam.



BEAM CRACK CHART
(SPAN 27.00')
A3-00-RB SOUTH

FIGURE A5-12



BEAM CRACK CHART
BEAM BO-OO-R NORTH
(SPAN 40.00')

NOTE: Heavy Lines Indicate Cracks at Failure of Beam.

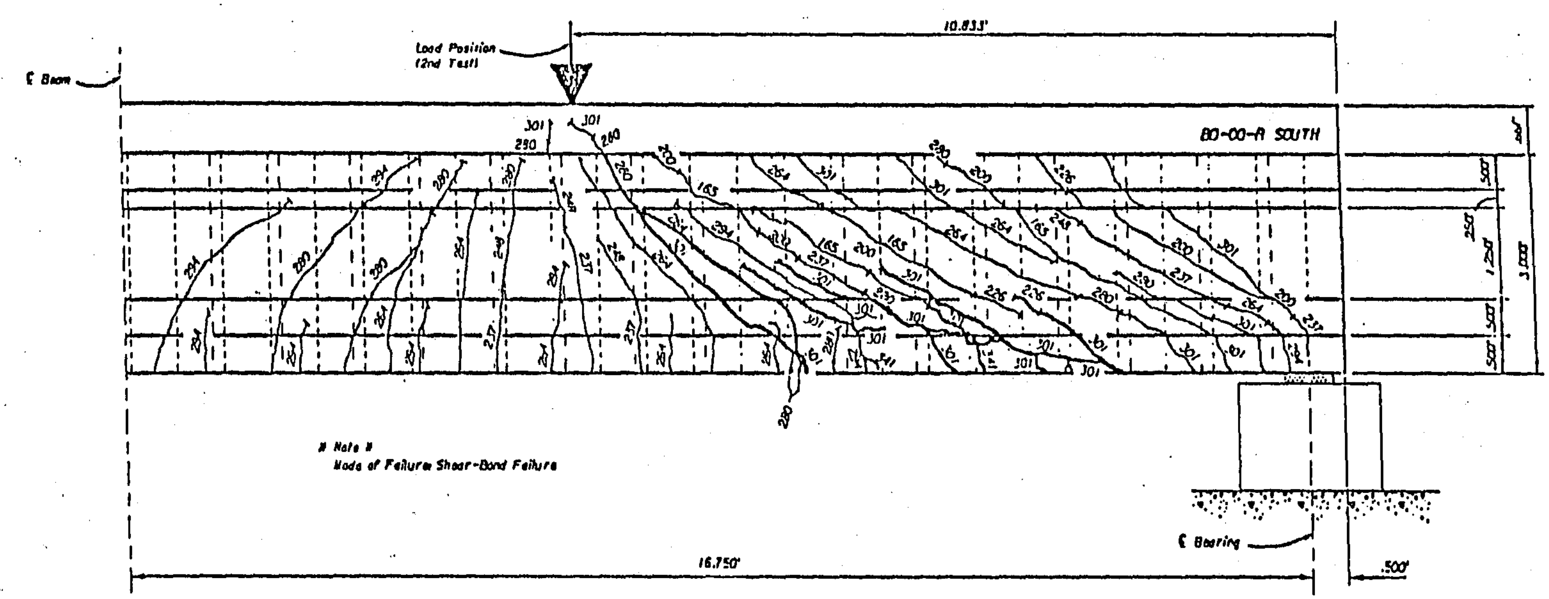
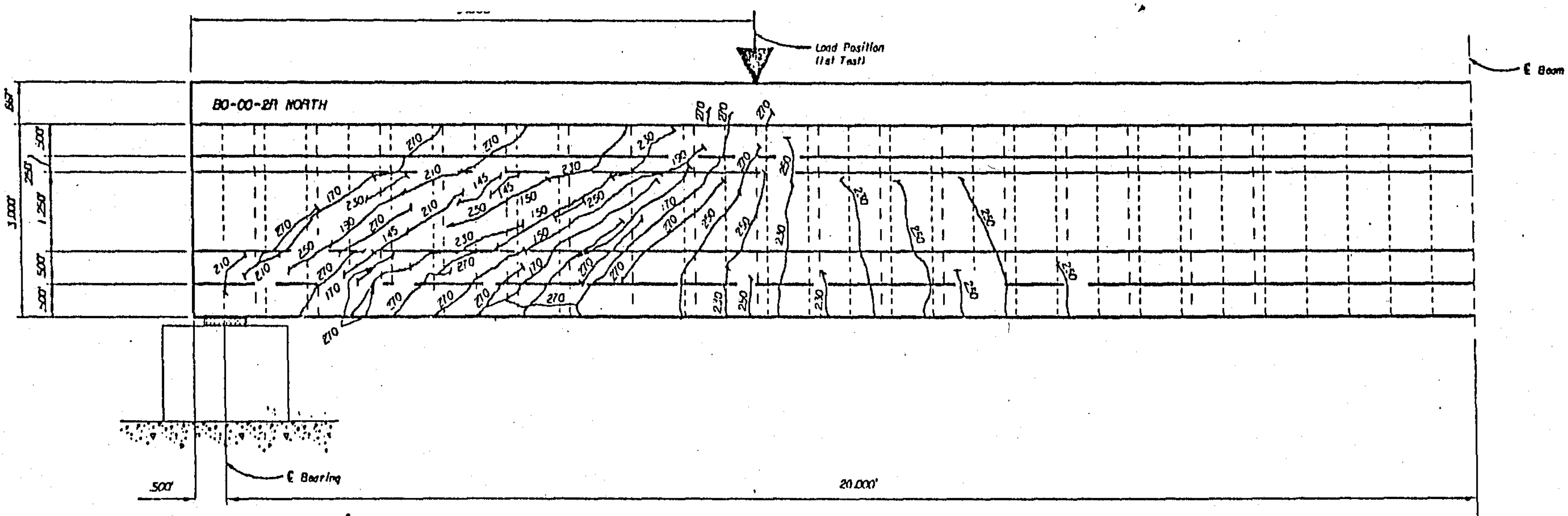
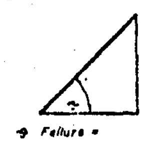


FIGURE A5-13

BEAM CRACK CHART
BEAM BO-OO-R SOUTH
(SPAN 31.50')





BEAM CRACK CHART
BEAM BO-00-2R NORTH
(SPAN 40.00')

NOTE: Heavy Lines Indicate Cracks at Failure of Beam.

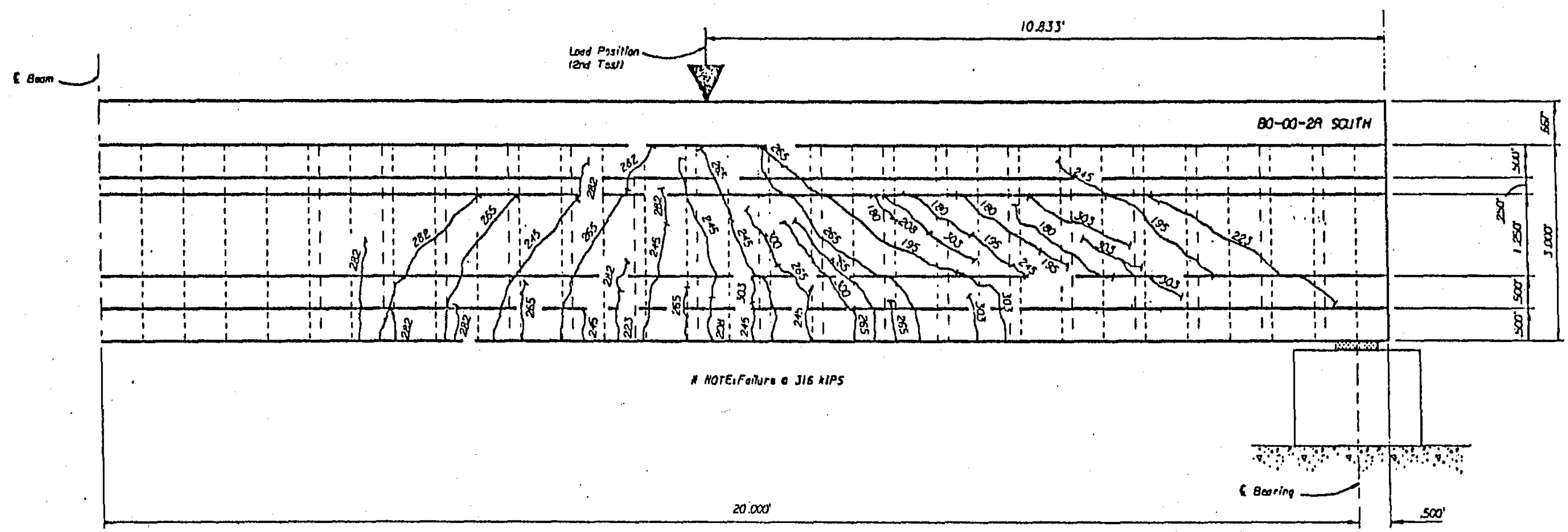
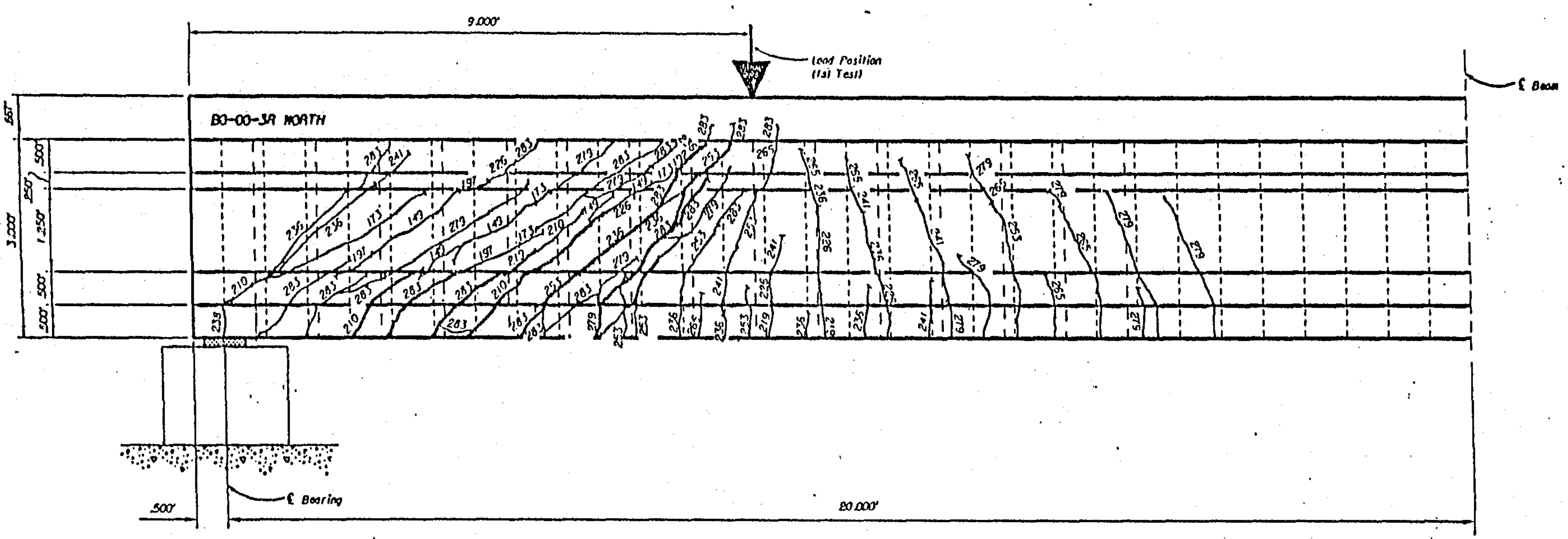


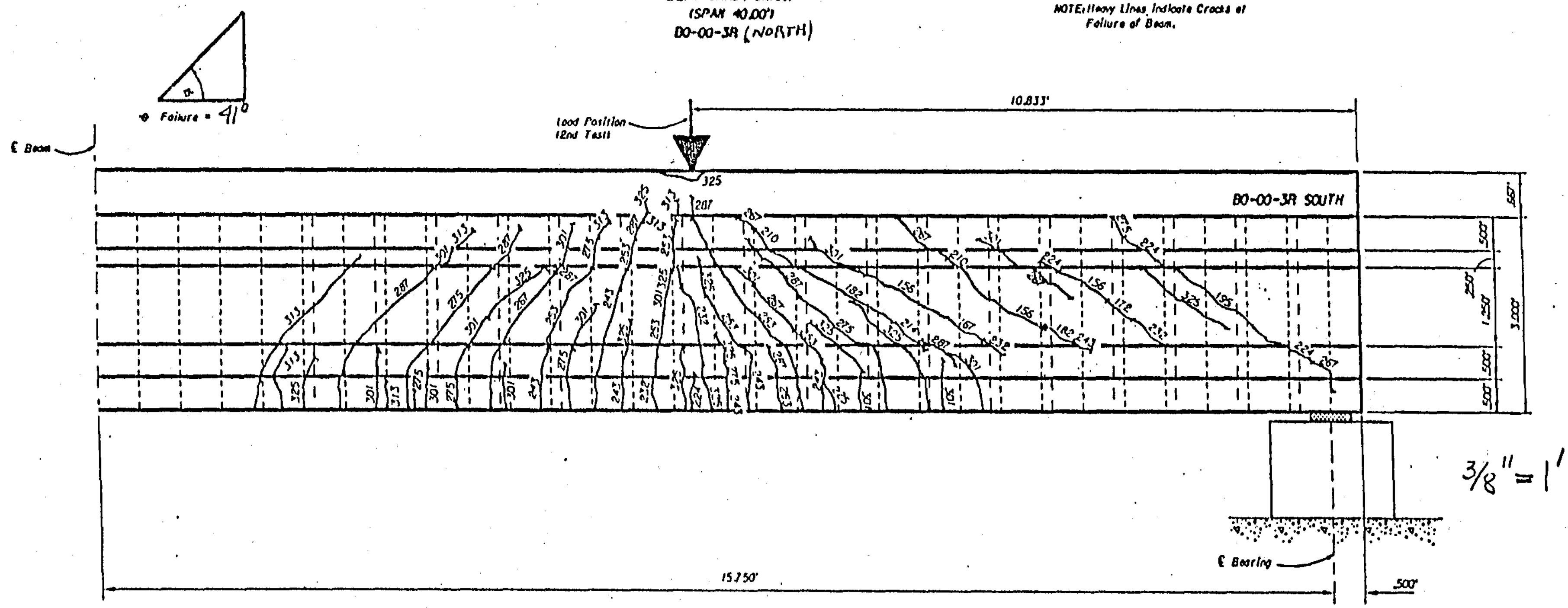
FIGURE A5-14

BEAM CRACK CHART
BEAM BO-00-2R SOUTH
(SPAN 31.50')



BEAM CRACK CHART
(SPAN 40.00')
BO-00-3R (NORTH)

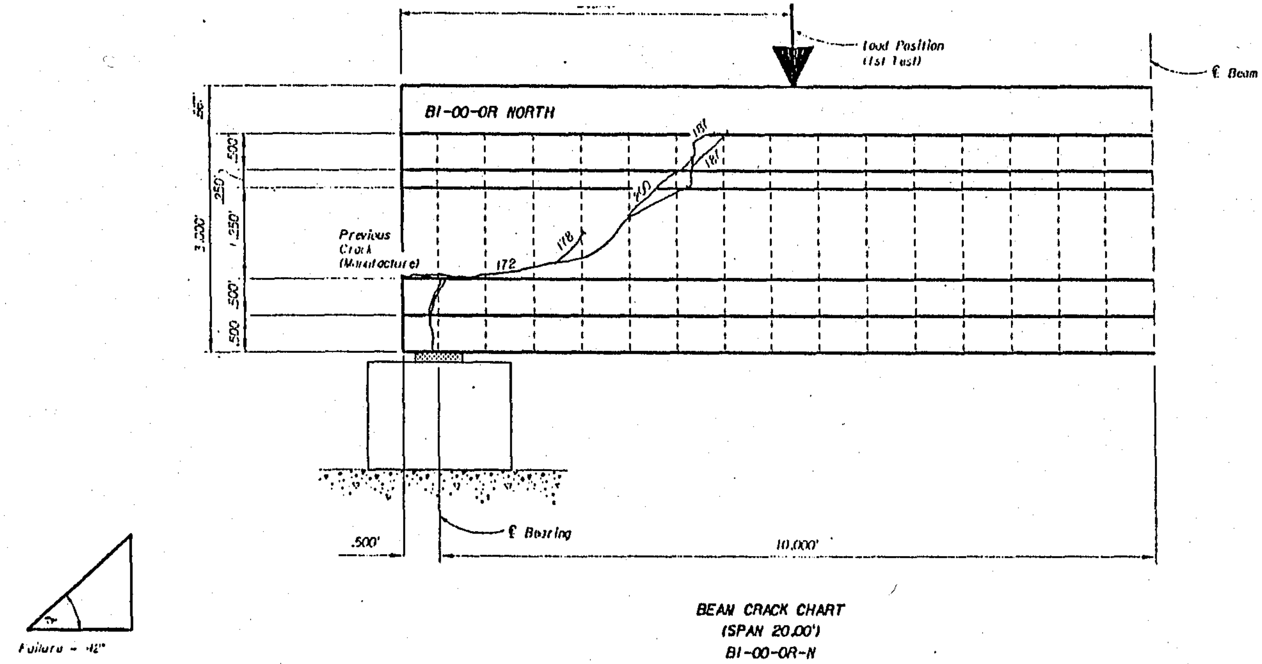
NOTE: Heavy Lines Indicate Cracks at Failure of Beam.



3/8" = 1'

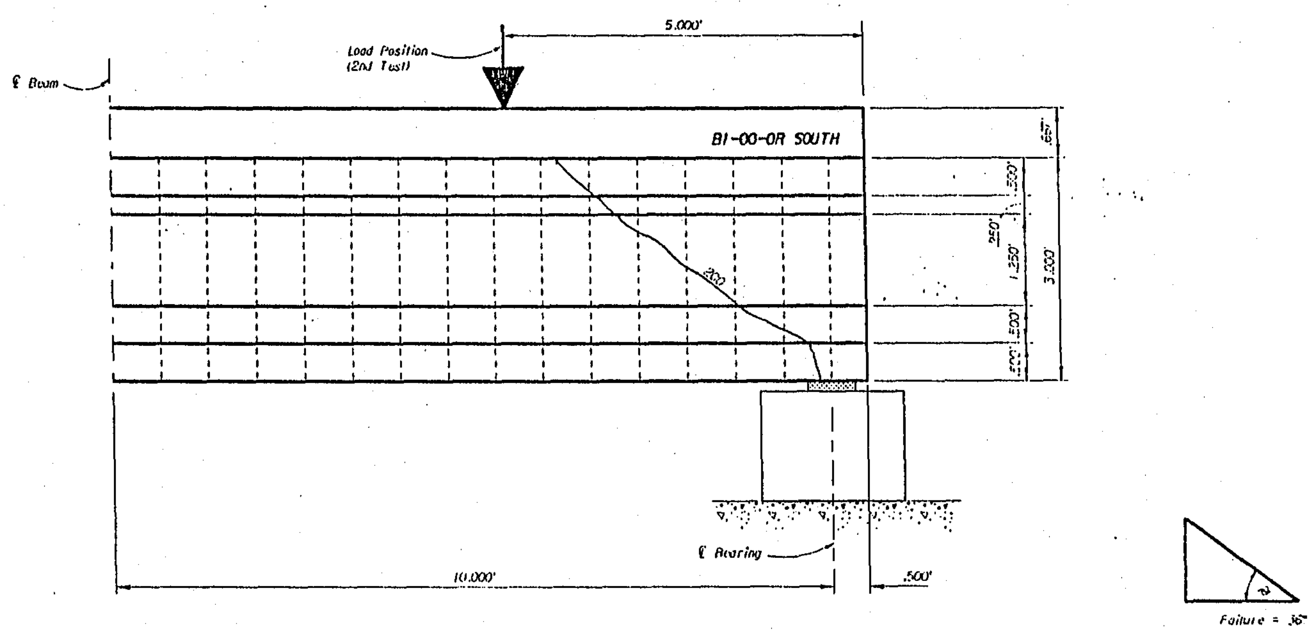
FIGURE A5-15

BEAM CRACK CHART
(SPAN 31.5')
BO-00-3R SOUTH



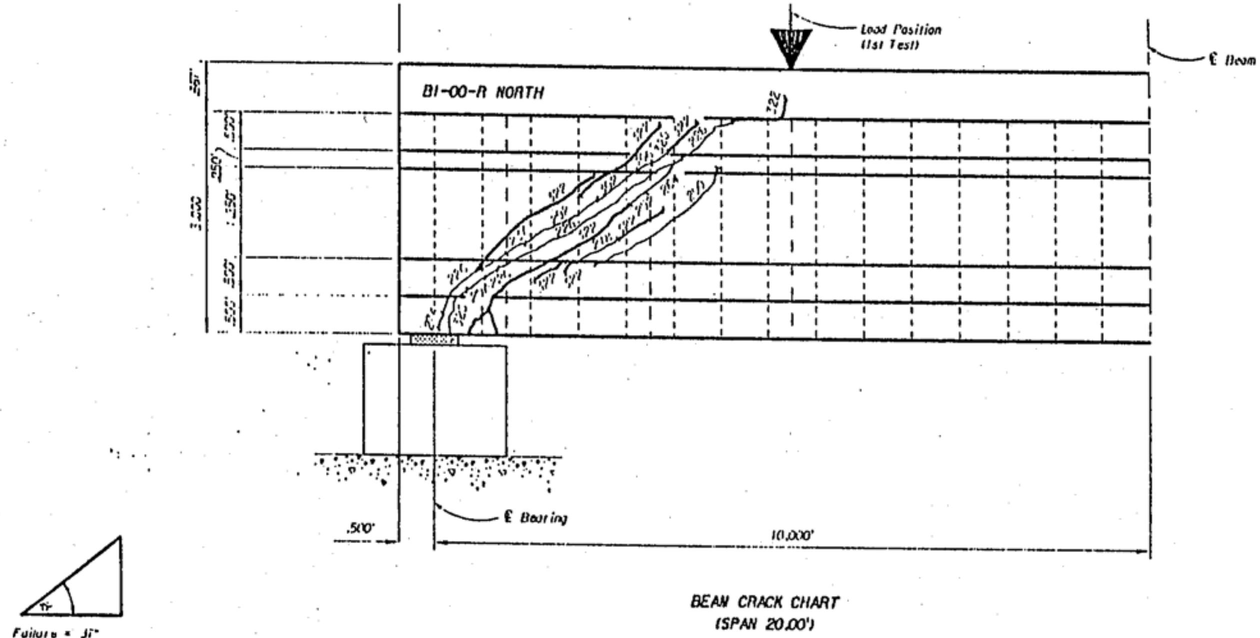
BEAM CRACK CHART
(SPAN 20.00')
BI-00-OR-N

NOTE: Heavy Lines indicate Cracks at Failure of Beam.



BEAM CRACK CHART
(SPAN 18.50')
BI-00-OR-S

FIGURE A5-16



BEAM CRACK CHART
(SPAN 20.00')
BI-OO-R NORTH
TEST DATE : 8-1-92

NOTE: Heavy Lines Indicate Cracks at Failure of Beam.

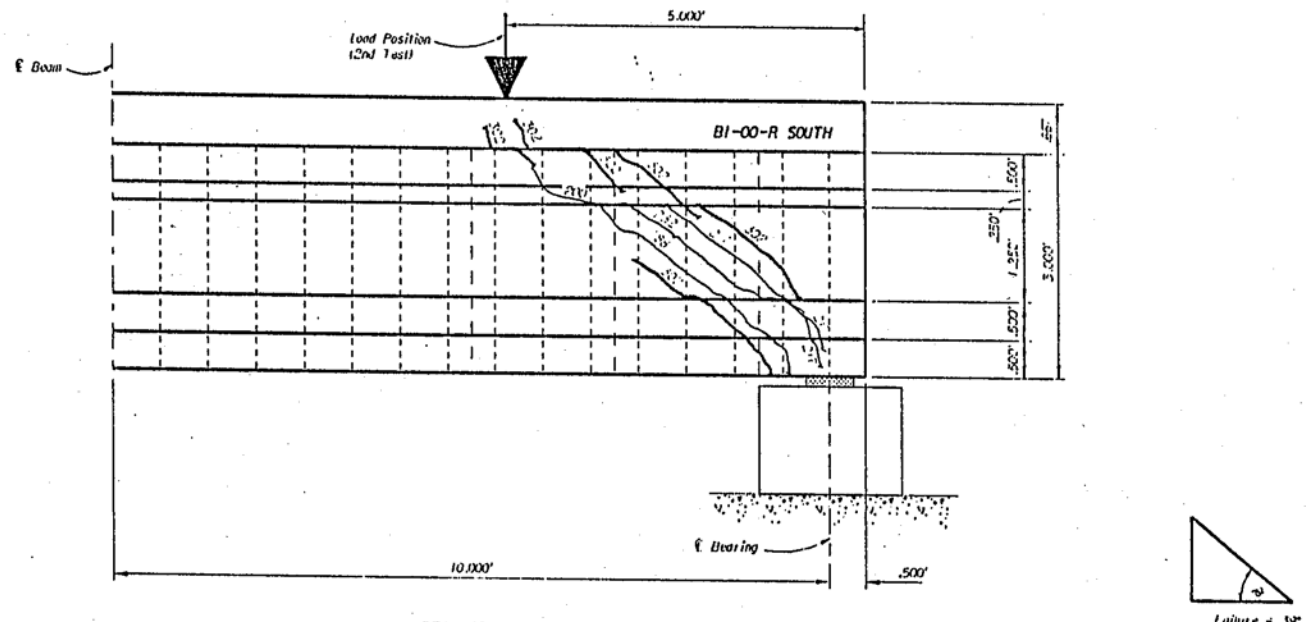
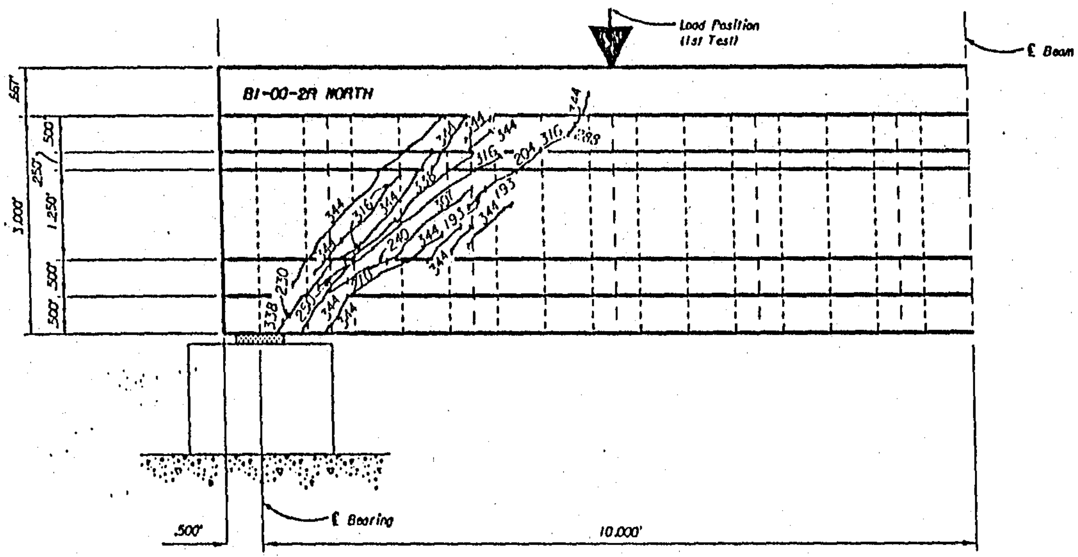


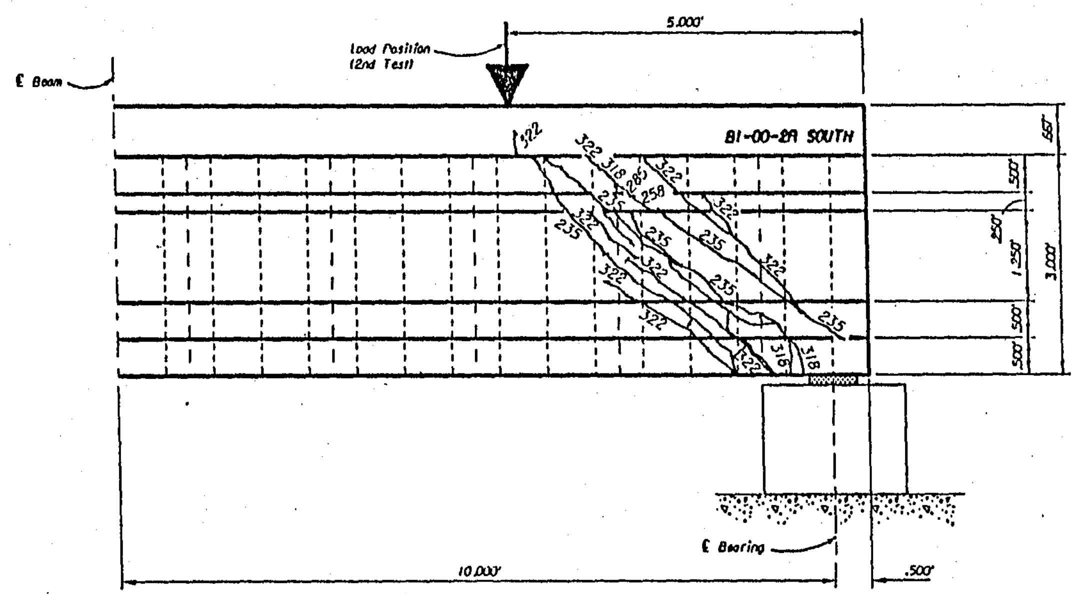
FIGURE A5-17

BEAM CRACK CHART
(SPAN 18.50')
BI-OO-R SOUTH
TEST DATE : 8-1-92



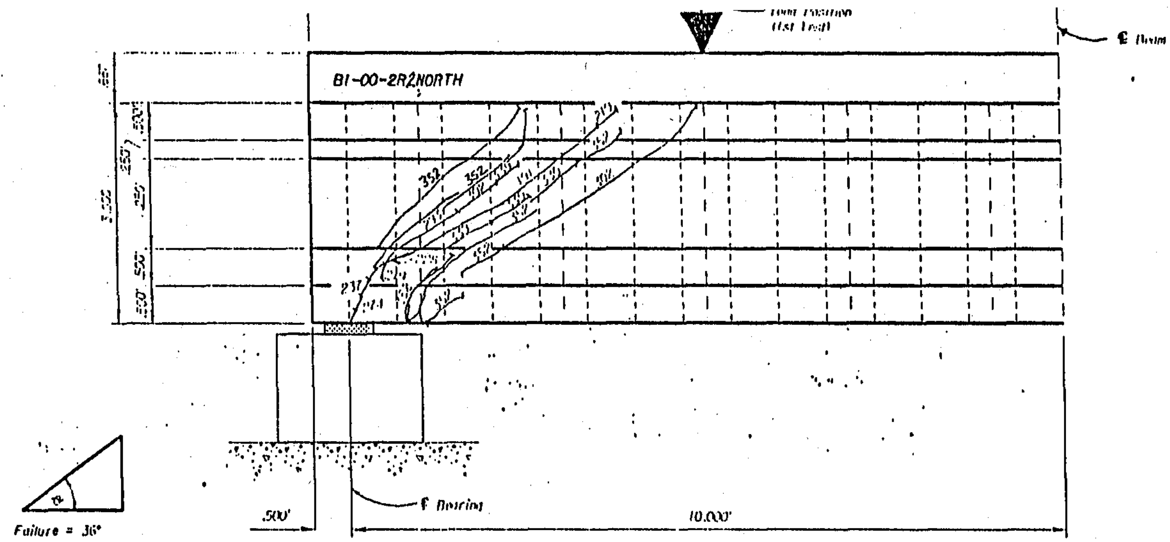
BEAM CRACK CHART
(SPAN 20.00')
BI-00-2R NORTH

NOTE: Heavy Lines Indicate Cracks at Failure of Beam.



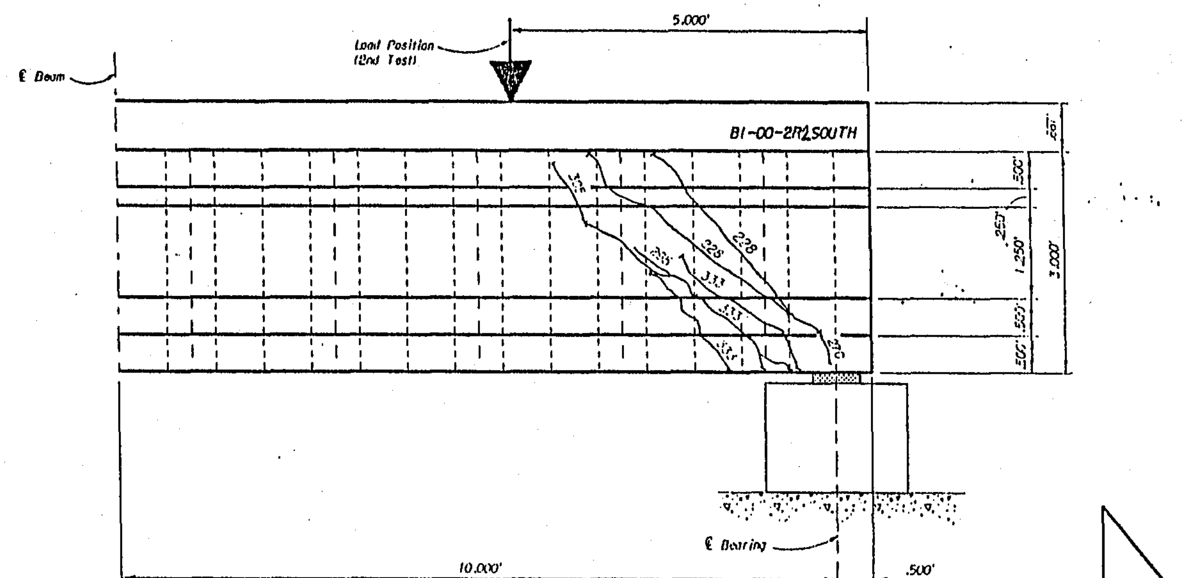
BEAM CRACK CHART
(SPAN 18.50')
BI-00-2R SOUTH

FIGURE A5-18



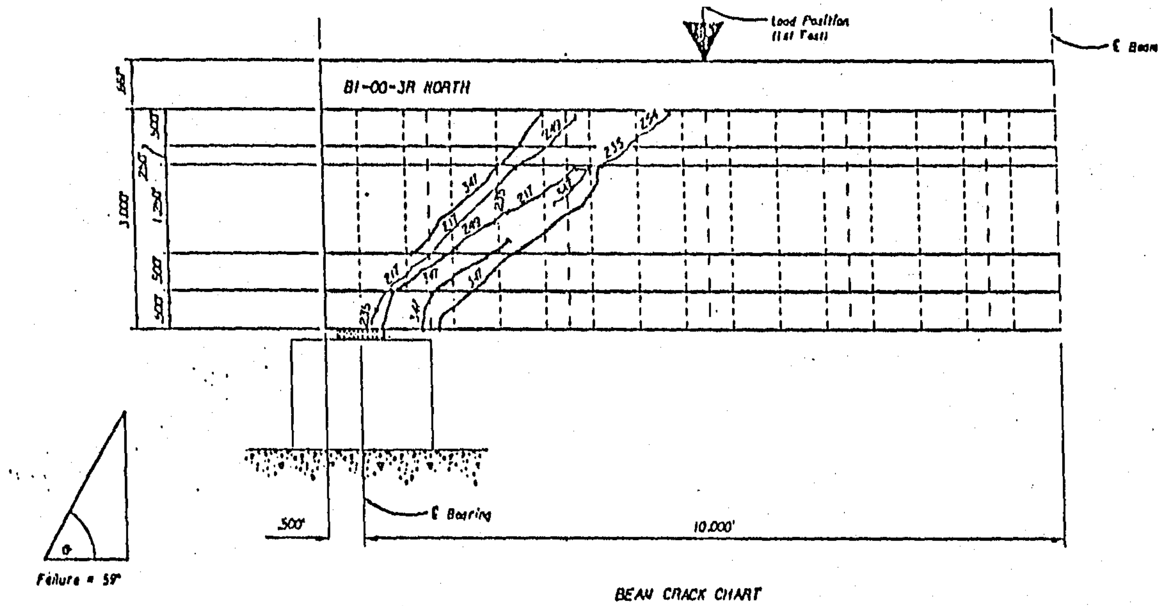
BEAM CRACK CHART
(SPAN 20.00')
BI-00-2R2NORTH

NOTE: Heavy Lines indicate Cracks at Failure of Beam.



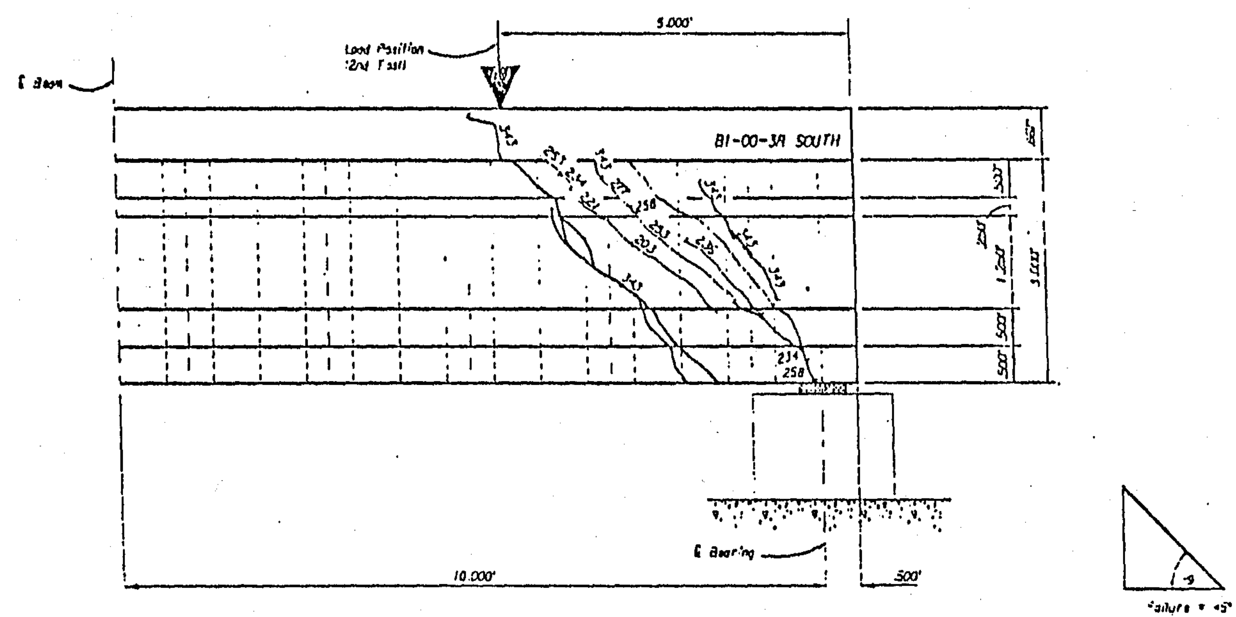
BEAM CRACK CHART
(SPAN 18.50')
BI-00-2R2SOUTH

FIGURE A5-19



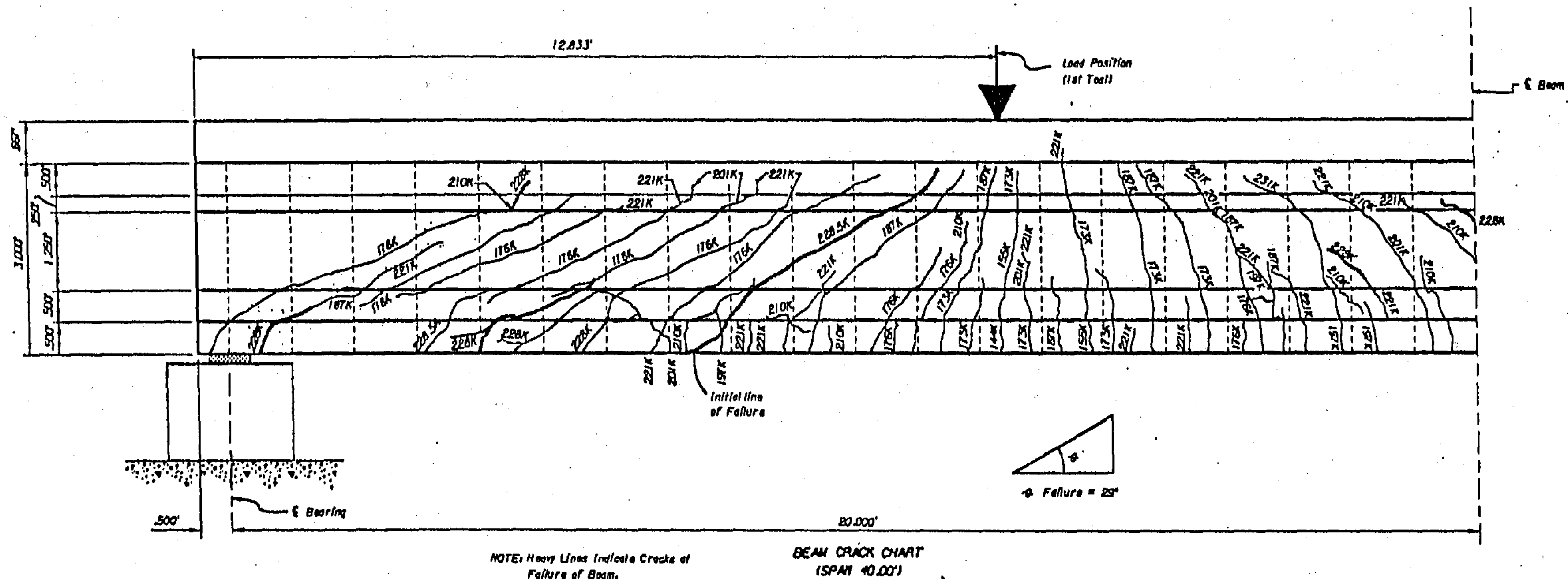
BEAM CRACK CHART
(SPAN 20.00')
BI-00-3R NORTH
TEST DATE: 8-19-92

NOTE: Heavy Lines Indicate Cracks at Failure of Beam.

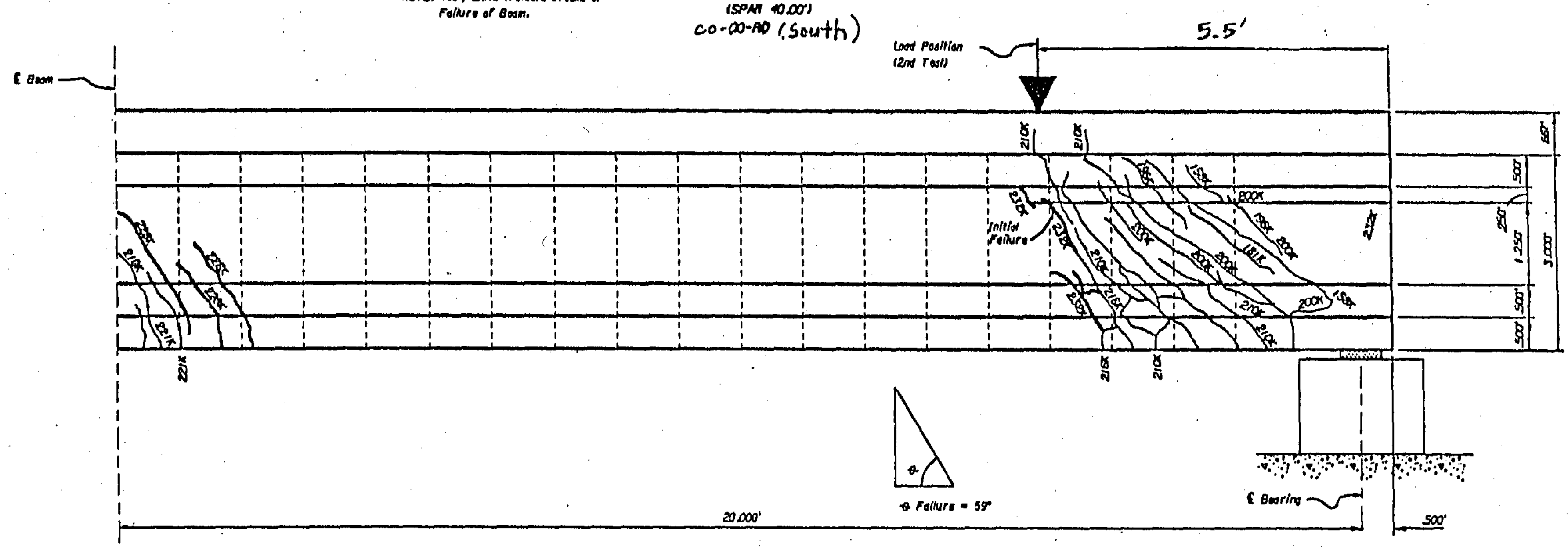


BEAM CRACK CHART
(SPAN 10.50')
BI-00-3R SOUTH
TEST DATE: 8-21-92

FIGURE A5-20

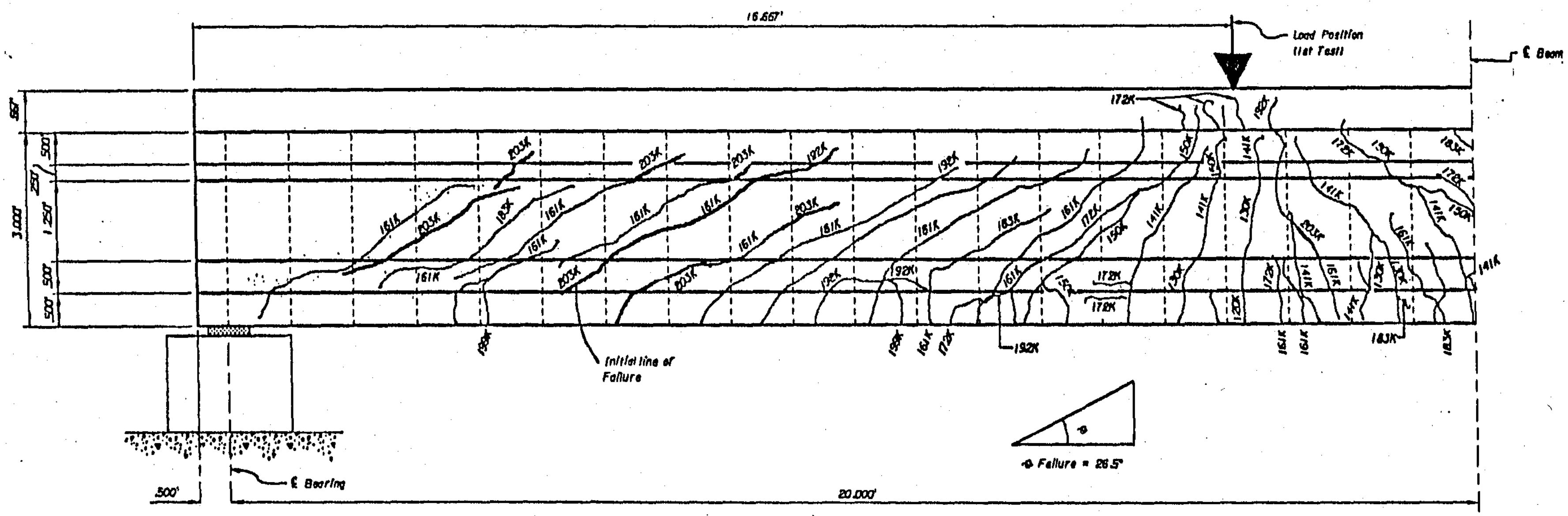


BEAM CRACK CHART
(SPAN 40.00')
CO-DO-RD (South)



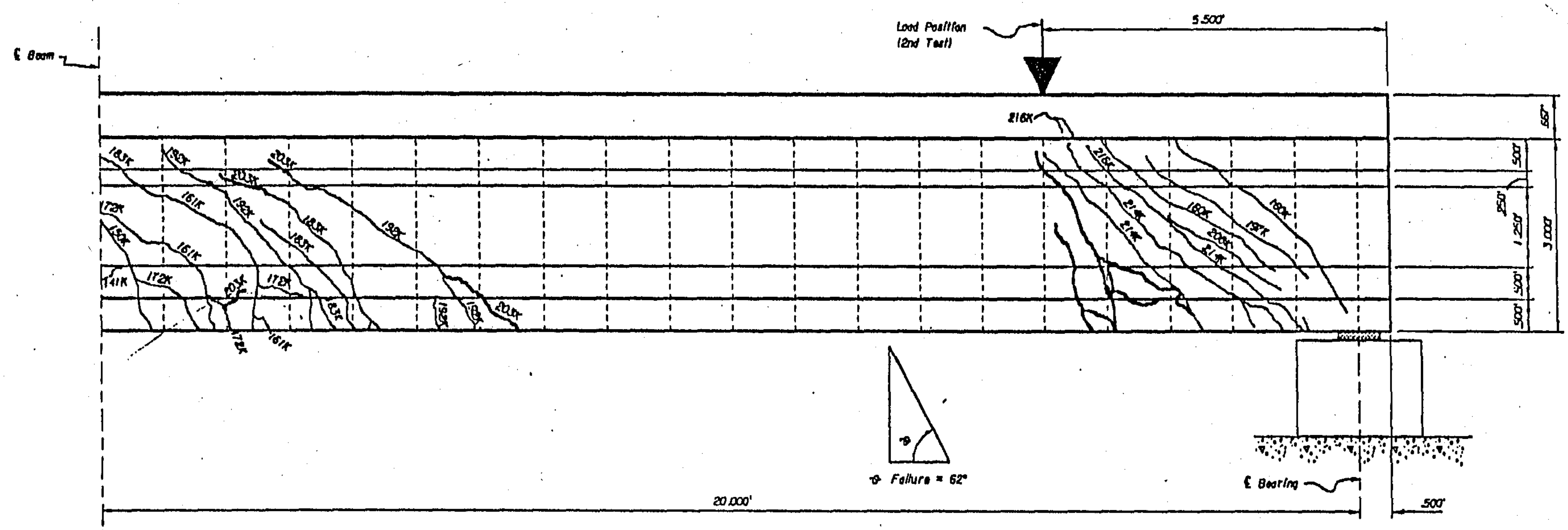
BEAM CRACK CHART
(SPAN 22.00')
CO-DO-RD (NORTH)

FIGURE A5-21



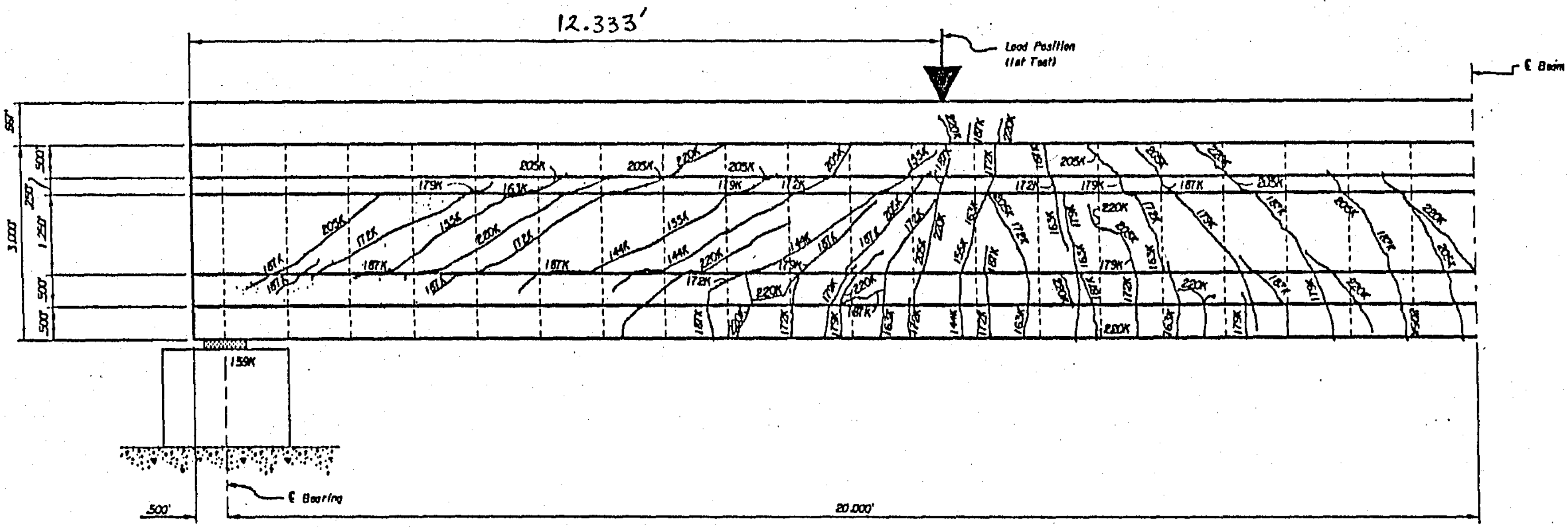
BEAM CRACK CHART
(SPAN 40.00')
CO-50-R (SOUTH)

NOTE: Heavy Lines Indicate Cracks at
Failure of Beam.

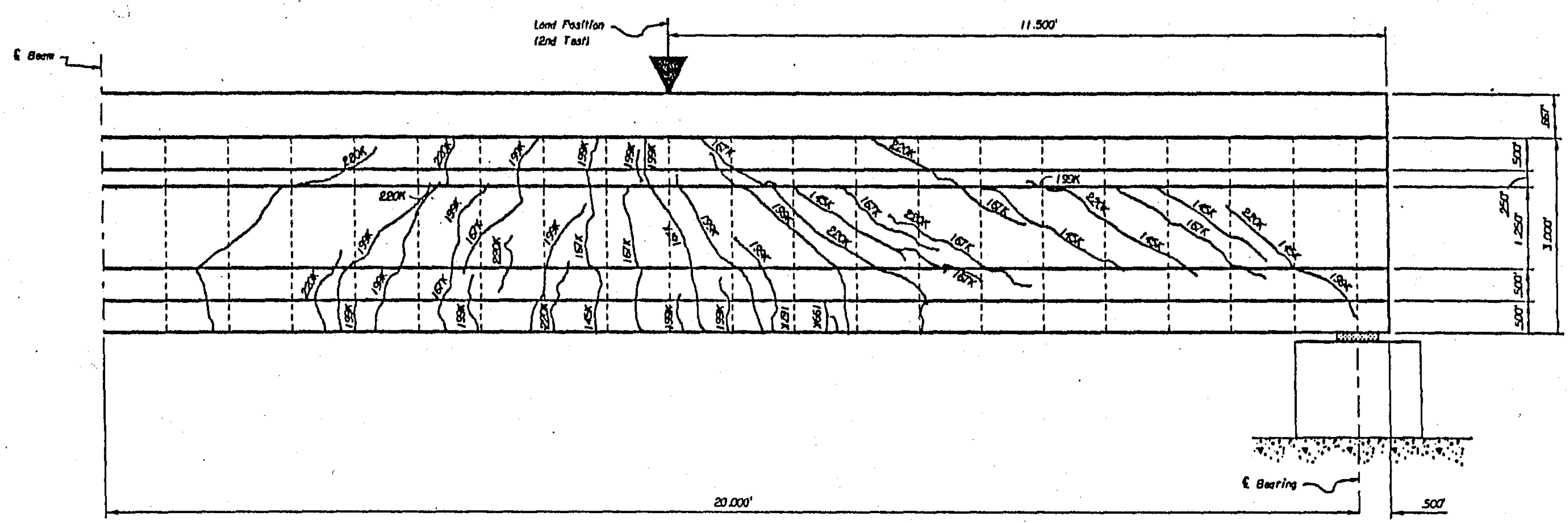


BEAM CRACK CHART
(SPAN 22.00')
CO-50-R (NORTH)

FIGURE A5-22

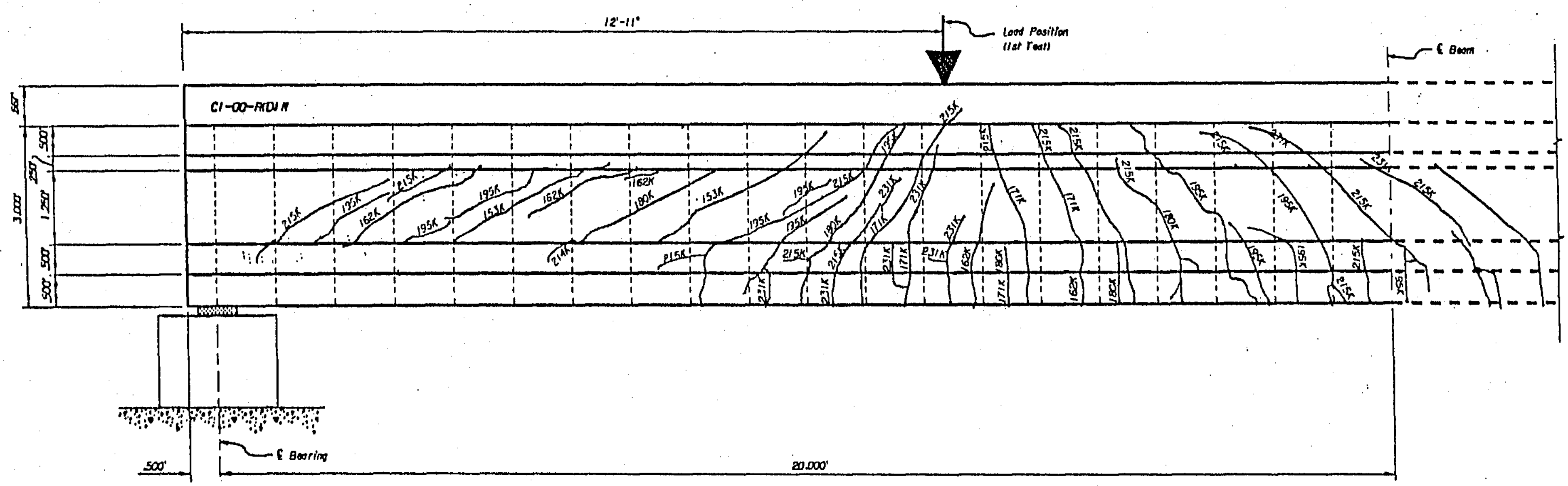


BEAM CRACK CHART
(SPAN 40.00')
CI-00 R (North)

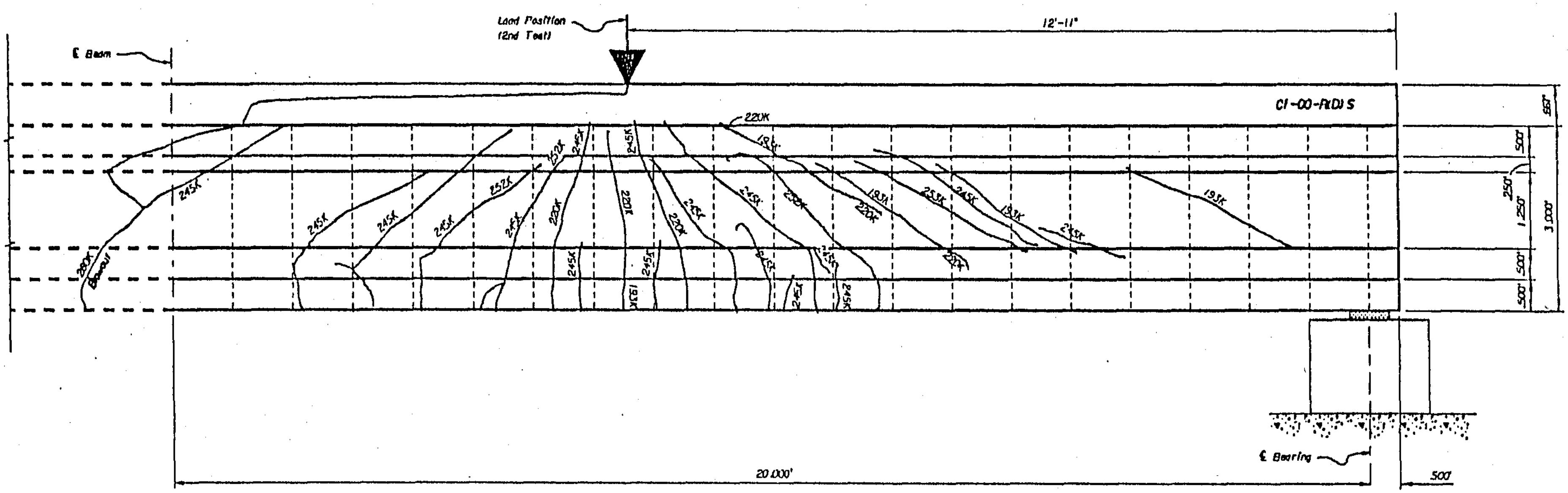


BEAM CRACK CHART
(SPAN 40.00')
CI-00 R (South)

FIGURE A5-23

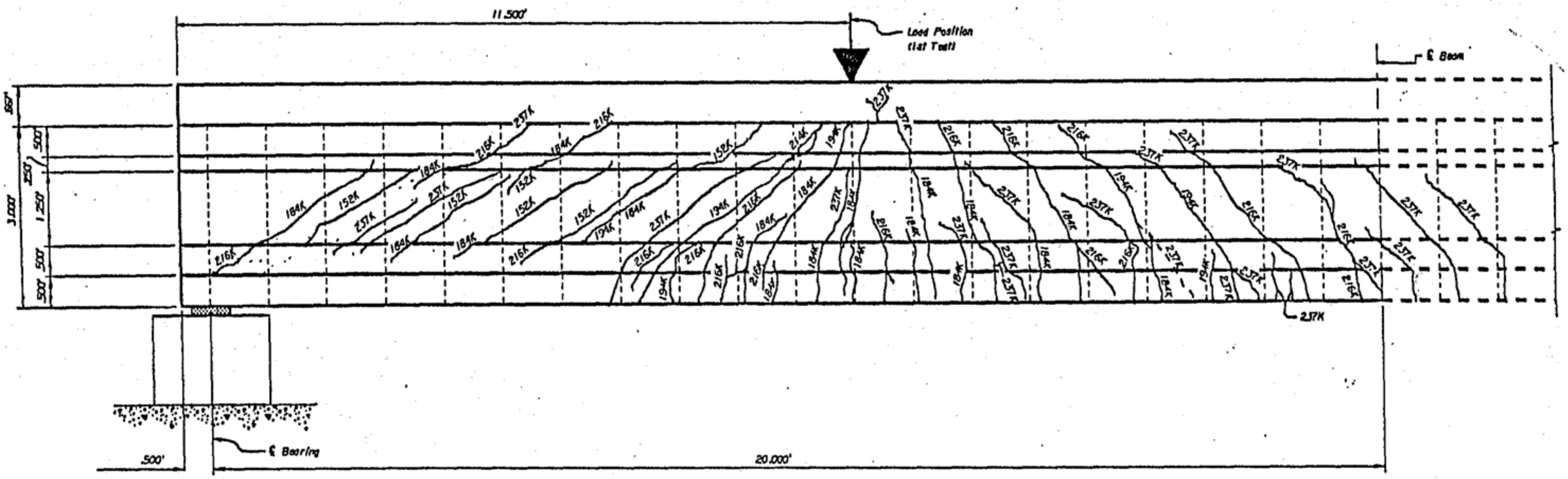


BEAM CRACK CHART
(SPAN 40.00')
CI-00-R(D) N

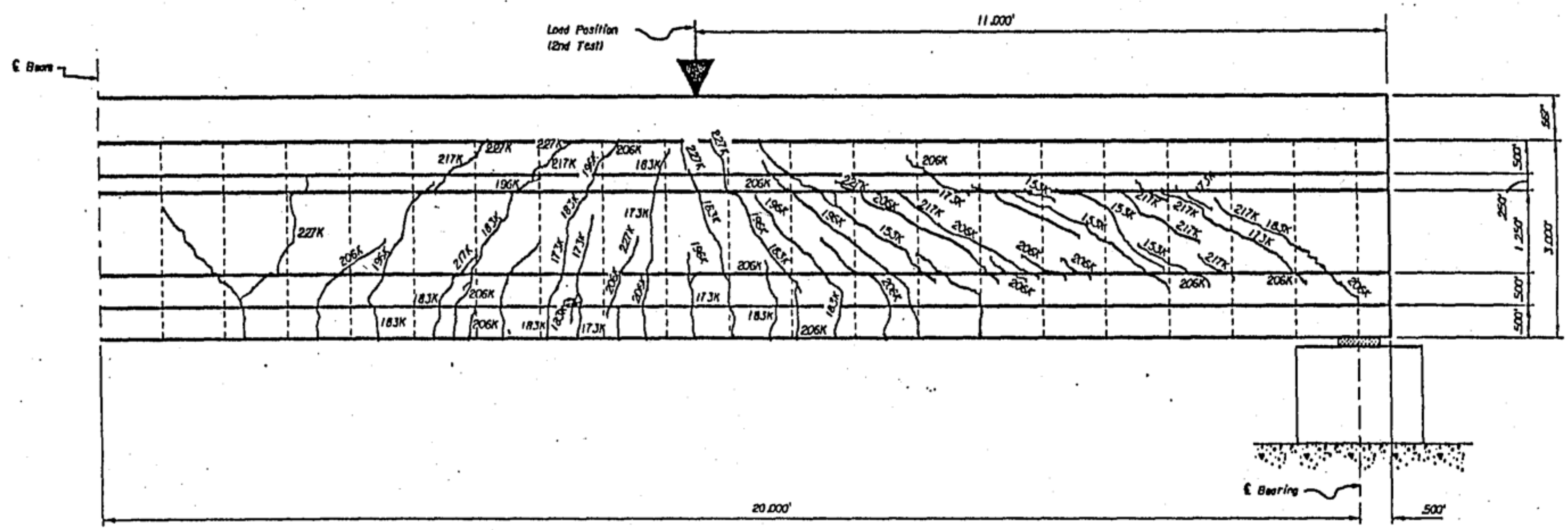


BEAM CRACK CHART
(SPAN 31.50')
CI-00-R(D) S

FIGURE A5-24

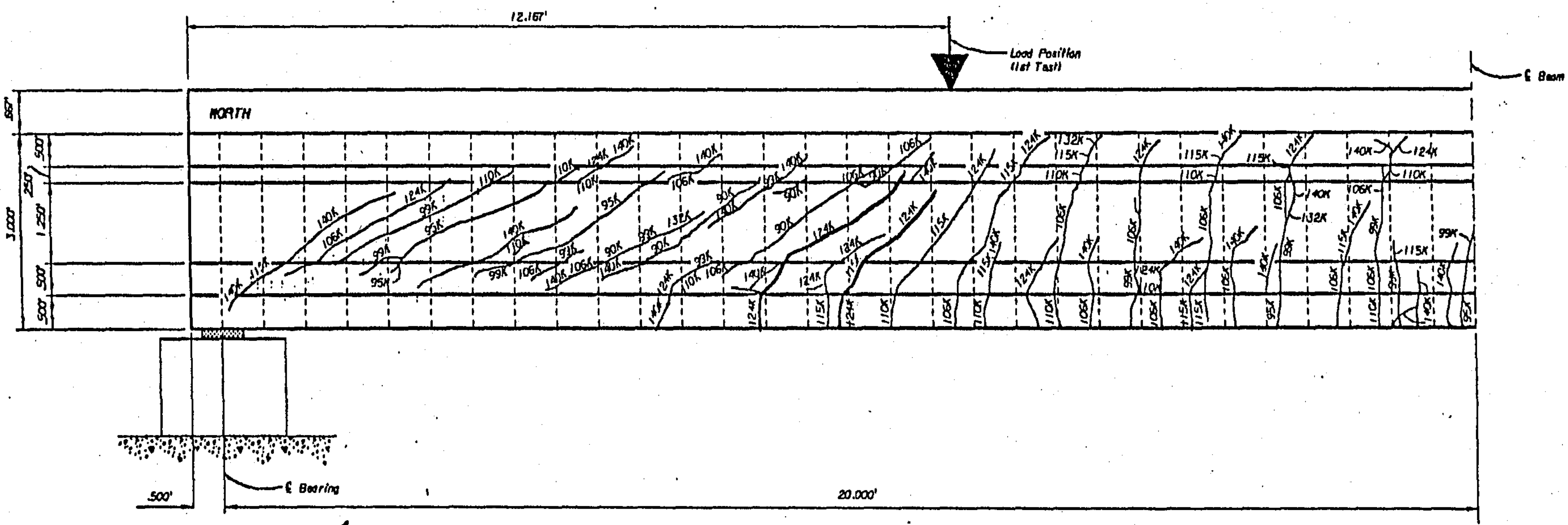


BEAM CRACK CHART
(SPAN 40.00')
CI-00-3R/2 (North)



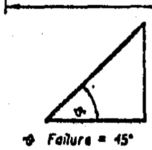
BEAM CRACK CHART
(SPAN 40.00')
CI-00-3R/2 (South)

FIGURE A5-25



BEAM CRACK CHART
(SPAN 40.00')
CI-25-R (North)

NOTE: Heavy Lines Indicate Cracks at Failure of Beam.



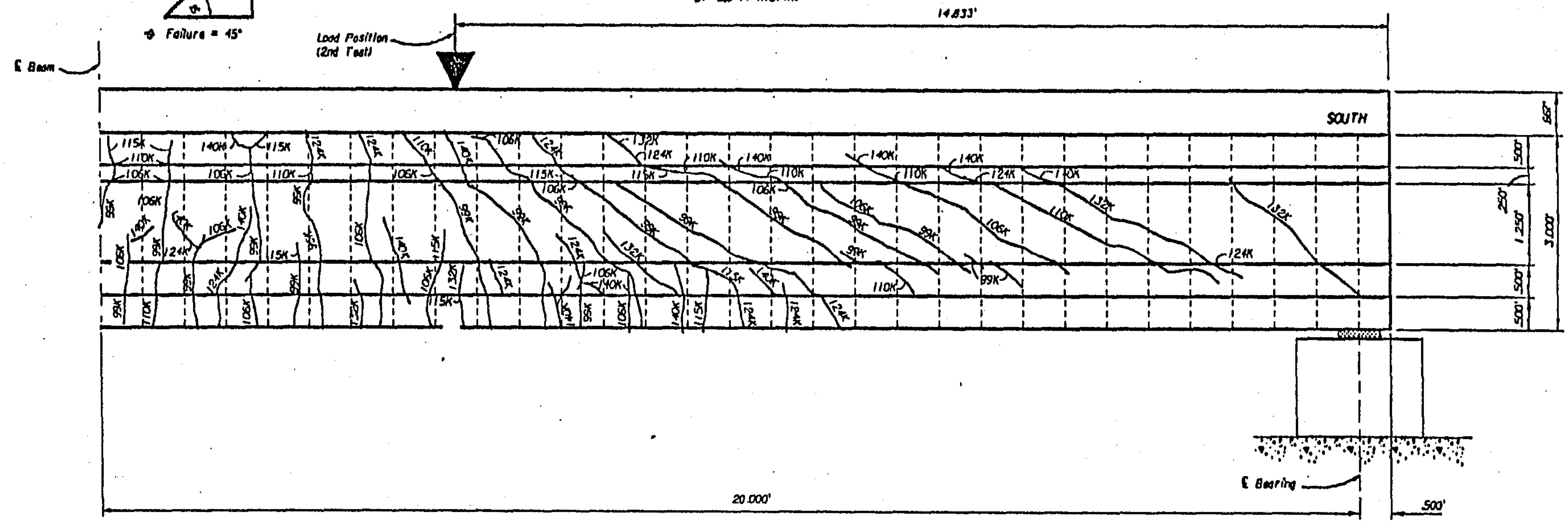
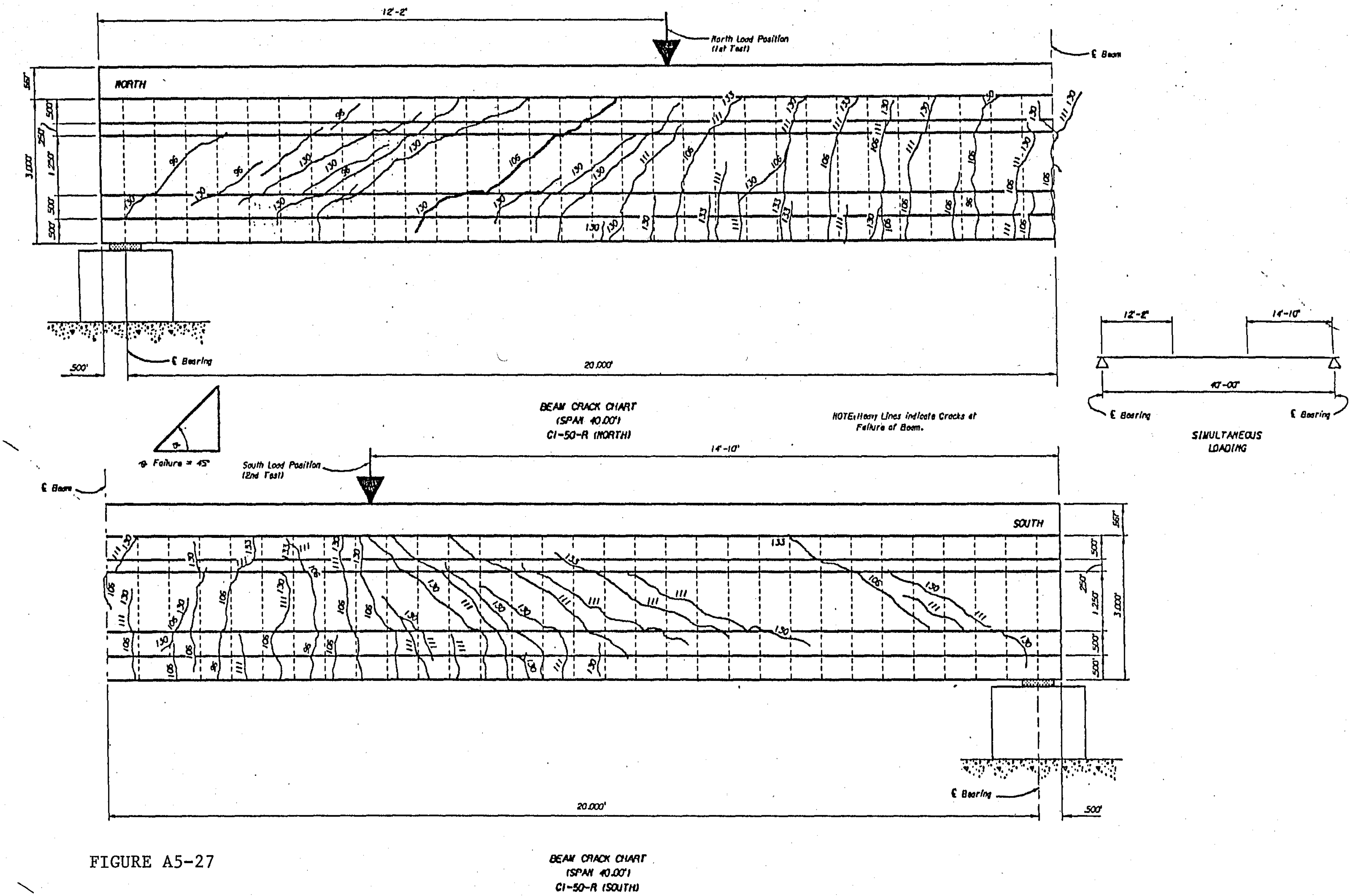


FIGURE A5-26

BEAM CRACK CHART
(SPAN 40.00')
CI-25-R (South)



12'-2"
North Load Position (1st Test)
℄ Beam
NORTH
℄ Bearing
20.000'
.500'
θ Failure = 45°
BEAM CRACK CHART
(SPAN 40.00')
CI-50-R (NORTH)
NOTE: Heavy Lines indicate Cracks at Failure of Beam.
12'-2"
14'-10"
40'-00"
℄ Bearing
℄ Bearing
SIMULTANEOUS LOADING
14'-10"
South Load Position (2nd Test)
℄ Beam
SOUTH
℄ Bearing
20.000'
.500'
BEAM CRACK CHART
(SPAN 40.00')
CI-50-R (SOUTH)

FIGURE A5-27