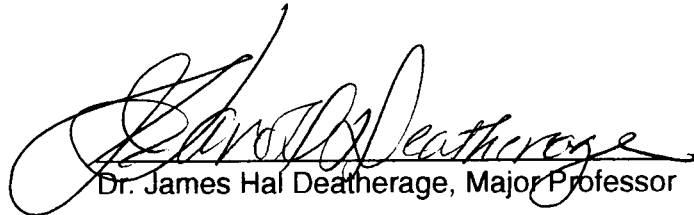


To the Graduate Council:

I am submitting herewith a thesis written by Jonathan Cyril Van Hook entitled "Computer-Simulated Flexural Tests of Fully Prestressed AASHTO Type-I Bridge Girders." I have examined the final copy of this thesis for form and content and recommend that it be accepted in partial fulfillment of the requirements for the degree of Master of Science, with a major in Civil Engineering.



Dr. James Hal Deatherage, Major Professor

We have read this thesis
and recommend its acceptance:

Edwin G. Burdette

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Accepted for the Council:



Associate Vice Chancellor
and Dean of The Graduate School

**COMPUTER-SIMULATED
FLEXURAL TESTS OF FULLY PRESTRESSED
AASHTO TYPE-I BRIDGE GIRDERS**

A Thesis

Presented for the

Master of Science

Degree

The University of Tennessee, Knoxville

Jonathan Cyril Van Hook

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ABSTRACT

The behavior of prestressed concrete beams is a function of several variables. Analytical methods have been developed for computing the ultimate behavior of prestressed concrete beams but at the high cost of experimentation. This study is an attempt to predict ultimate behavior of AASHTO Type-I prestressed beams through the finite element program, ABAQUS. A review of the existing analytical and experimental studies are also included.

The results obtained from ABAQUS were compared to experimental results (UTK). The effective prestress, the concrete strain, and the concrete strength was varied. Load-deflection plots, steel stresses and strains, and concrete stresses and strains were obtained.

The major factor affecting the behavior at cracking was determined to be the effective prestress. Ultimate deflections were governed by ultimate concrete strain. A change in the effective prestress caused no appreciable change in the nominal moment capacity.

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LIST OF SYMBOLS

<u>SYMBOL</u>	<u>DESCRIPTION</u>
A_s	Area of tensile steel (in^2)
A_s'	Area of compression steel (in^2)
C_b	Distance from bottom of member to neutral axis (in)
c	Depth of compression zone at M_{ult}
d_p	Depth to the centroid of the bottom prestressed steel strand
d_p'	Depth to the centroid of the top prestressed steel strand
E_c	Modulus of elasticity of concrete (ksi)
E_s	Modulus of elasticity of steel (ksi)
ϵ_2	Longitudinal strain corresponding to S_2
ϵ_{ce}	Compressive strain in concrete at level of steel due to effective prestress force
ϵ_{cu}	Compressive strain in concrete at level of steel at ultimate moment
ϵ_{se}	Steel strain due to effective prestress
ϵ_{su}	Ultimate steel strain at location of maximum moment
ϵ_u	Ultimate compressive strain of concrete
F	Change in steel strain divided by change in concrete strain
f'_c	Compressive strength of concrete (ksi)

<u>SYMBOL</u>	<u>DESCRIPTION</u>
f_{su}	Stress in the strand at nominal strength (ksi)
f_{se}	Effective stress in the strand after losses (ksi)
f_t	Tensile stress of concrete
I	Moment of inertia (in^4)
k_1	Ratio of average compressive stress to the maximum compressive stress of the concrete in the compression zone of the beam at ultimate
k_2	Ratio of the depth to the line of action of the resultant compressive force in the compression zone to the depth to the neutral axis
k_3	Ratio of maximum compressive strength of the concrete in flexure to the cylinder strength
$k_1 k_3$	Ratio of the average stress in the compression zone at failure to the cylinder strength
k_u	c/d or $q_u/k_1 k_3$
M_{ult}	Ultimate Moment (kip-ft)
ρ	A_s/bd
Q'	$E_s \rho / k_1 k_3 f'_c$
q_u	$\rho f_{su} / f'_c$
S_2	Stress corresponding to 40 percent of the ultimate stress
S_1	Stress corresponding to a strain of .00005

Chapter I

Introduction

An acceptable effective prestress force in concrete was not accomplished until the introduction of high strength steel wires. High strength-steel wires enable stress to be introduced without a large relative loss of strain in the steel wires due to creep and shrinkage of the concrete. In 1928, Freyssinet of France bonded concrete to prestressed high-strength wires. His efforts were successfully followed by Hoyer of Germany who developed the first effective means of prestressing by stretching high-strength wires between two buttresses, pouring and curing the concrete, and cutting the wires after the concrete had hardened.

The initial prestress load plays a substantial role in both the behavior and serviceability of the member. Prestressing enables a member to carry higher service loads without cracking the concrete. This is important because cracked sections cause premature weathering of the steel which affects the ultimate behavior of the member.

Since Freyssinet, researchers have been developing analytical methods for predicting both the cracking behavior and ultimate flexural behavior of prestressed concrete products. Because of the linear-elastic behavior up to the cracking state, the behavior up to the cracking state is well understood. The inelastic response of prestressed beams, however, is still not fully understood. Variables such as the concrete strength and the steel ratio affect the inelastic response and ultimate behavior of

prestressed beams. Two conditions which have been investigated are over-reinforced and under-reinforced. The strength of an over-reinforced beam depends on the amount of reinforcement and is particularly sensitive to the strength of the concrete, while the strength of an under-reinforced beam depends primarily on the strength of the steel. The current 318-89 ACI Code does not permit the design or construction of over-reinforced concrete members due to sudden failures. Rational analyses, using the equilibrium of forces and strain compatibility, have been developed but at the high cost of experimentation. Methods, such as the finite element method, need to be explored for predicting the ultimate capacity of prestressed beams.

The objective of this study is to: (1) investigate some of the experimental and analytical research on the ultimate strength of prestressed beams, (2) predict the behavior of AASHTO Type-I beams using the finite element program, ABAQUS, (3) compare the results with test results of experiments at The University of Tennessee at Knoxville, and (4) predict the ultimate strain in the concrete via ABAQUS.

Chapter II

Literature Review

The definition of prestressed concrete proposed by ACI-ASCE committee 423 is "Concrete in which there have been introduced internal stresses of such magnitude and distribution that stresses resulting from given external loadings are counterbalanced to a desired degree [10]"

Prestressing concrete members was accomplished in 1928 when Freyssinet, the father of prestressed concrete, bonded high-tensile steel bars and high-tensile steel wire (piano wire) to concrete. The high tensile strength of the wires, with ultimate stresses as high as 350 ksi, produced large concrete compressive stresses while maintaining high effective steel strains after losses due to creep and shrinkage in the concrete [6,12]. High strength wires greatly reduced the percentage of steel needed for prestressing when compared to high strength bars because the allowable stresses in the wires were approximately 3 times that of high strength bars [6].

Evans [6] conducted tests on both bar and wire beams from an initial prestress of zero to 25 tons. The major conclusions of the studies were as follows:

- (1) The cracking load of the prestressed bar beams was approximately one-half that of the wire reinforced beams. This behavior is due to the large prestress loss in the bar beams due to strain, creep, and shrinkage of the concrete [6].

- (2) The wire reinforcement was extremely efficient in both the cracking and ultimate behavior of the beam. Calculations using the Bernoulli-Euler theory proved to be very effective in predicting the concrete stresses and the cracking loads [6].

The research by Evans was followed by that of Abeles [1]. Abeles notes that in a prestressed cracked section, the tensile zone is greatly reduced when compared to a regular reinforced section. If the cracks do not widen to promote any permanent elongation of the steel, the cracks will close upon the removal of the load and the compressive stresses will be transmitted between the adjacent regions of the cracked section. This behavior is altered when wide cracks occur which lead to the plastic deformation of the steel.

Abeles and K. Hajnal-Konyi conducted preliminary tests on beams reinforced with high tensile wire. The objective of the experiment was to compare the behavior of three identical beams with different prestress conditions; (a) not tensioned, (b) 40 percent prestressed, and (c) fully prestressed. Two beams of each type were to be made simultaneously for each prestress.

The beams were 7 feet long with a clear span of 6 feet and two loads were applied at the third points of the 6 foot span. The deflection was measured at mid-span with a dial micrometer. Crack formations were noted and the crack widths and depths were measured under various loads.

Abeles concluded that the ultimate tensile resistance of the concrete was not increased by prestressing, but the resistance against cracking and the resilience of the structure after cracking was greatly improved [1]. Results obtained could not be used in a comparative study (i.e. ultimate moment capacity) because two different concrete strengths were used.

Tests by Hoyer demonstrated that the initial prestress force does not change the ultimate capacity of the beam appreciably [1,7]. Hoyer tested four series of beams of the same cross section. Each beam was reinforced with high strength wire (369,000 psi) with different initial prestress (14,220 psi, 56,880 psi, 113,760 psi, and 170,640 psi). The ultimate load in each beam was approximately the same. Two beams in each of the four cases were tested simultaneously. In all cases, the theoretical steel stress at failure was 330,000 psi or 89.4 percent of the ultimate strength of the wire [1,7].

Revesz [6] distinguishes between three different modes of failure for prestressed beams (not including failure due to diagonal tension, horizontal shear, or bond failure). These modes are:

- (1) Fracture of the steel in tension.
- (2) Excessive elongation of the steel, followed by the rise of the neutral axis, causing the crushing of the concrete in the compression zone which is the mode of failure. The failure is characterized by wide cracks in the tensile zone and large

deflections prior to failure. An increase in the steel area results in an increase in the ultimate moment.

- (3) Primary compression failure which occurs in over-reinforced beams. The failure is characterized by the sudden collapse of the concrete in the compression zone without warning by cracking or deflection, due to the limited elongation of the steel. The addition of further tensile steel or increase of prestressing force does not result in an appreciable increase in the ultimate bending moment.

Beams which failed by fracture of the steel or excessive elongation of the steel were considered to be under-reinforced. The mode of failure in the prestressed beams was determined by the condition of the bond between the steel and the concrete and the effective prestress in the steel.

Eleven, 14 foot span, beams were tested and split into two groups. One was precast units prestressed on the Hoyer system, and the second was similar pre-tensioned beams with in-situ concrete forming T-beams. Prestress, percentage of steel, and the quality of the in-situ concrete was varied. The pre-tensioned beams were 3 in. wide by 6 in. deep with 12 gauge wire (ultimate strength = 120 tons/in²). The composite beam depth was 8 in. with a 21 1/2 inch wide by 1 1/2 inch thick flange.

The average ultimate bending moment for the first group was 53.4 ton-in. with a maximum deviation of 9 percent. The average ultimate bending moment for the second group was 103.0 ton-in. with a

maximum deviation of 9 percent. The variations in the moment were caused by a change in the depth of the moment arm. The data in Table 2.1 indicates that the effective prestress force did not have an appreciable effect on the ultimate bending moment. The increase in the construction depth of the two groups, from 6 in. to 8 in., produced an ultimate bending moment ratio which was on the order of two. T-beams L and F produced the same ultimate moment capacity. They both had two very different ultimate concrete strengths and only slightly different moment arms. All beams, except for beam E, failed by fracture of the bottom steel [16].

Revesz drew the following conclusions based on an examination of the results [16]:

- (1) In pre-tensioned beams, the ultimate strength of the steel may be developed provided the bond is good.
- (2) All the beams tested displayed a large number of wide cracks and gave adequate warning of impending collapse by large deflections.
- (3) The use of low-grade in situ concrete in the flange of composite T-beams does not have a detrimental effect on the ultimate bending moment capacity of the beam, provided the flange is relatively large.

Table 2.1 - Results of tests on beams reinforced with 12 wires near the soffit.

Mark	GROUP I					GROUP II				
	A	B	H	E	M	L	G	F		
d_b (in)	4.605	4.845	4.77	5.00	6.956	7.46	7.01	7.74		
d_t (in)	0.55	0.55	0.55	0.55	2.563	3.35	2.62	4.00		
A_b (sq. in.)	0.1044	0.1044	0.1044	0.1044	0.1044	0.1044	0.1044	0.1044		
A_t (sq. in.)	0.0174	0.0174	0.0174	0.0174	0.0174	0.0174	0.0174	0.0174		
f'_c (psi)	7,820	7,820	7,840	7,650	2,240	6,560	5,560	2,280		
μ (ton-in)	49.42	55.30	57.80	51.38	93.32	107.88	99.48	107.88		
f_{se}/f_{su}	0.60	0.60	0.35	0.00	0.60	0.60	0.35	0.00		
f_{se} (ton/sq.in)	78.2	78.2	48.9	0.0	80.6	80.6	48.9	0.0		

(Source: Revesz, S., "Factors Governing the Ultimate Bending Moment of Normal Reinforced and Prestressed Concrete Beams, with Reference to a Proposed Plastic Theory," *Magazine of Concrete Research* (London), No. 13, Aug. 1953, pp. 17, 25.)

- (4) In beams with untensioned wires, it is possible to reach the ultimate strength of the steel (beam F) where the percentage of steel is relatively low.
- (5) In beams with untensioned wires and with a relatively high percentage of reinforcement, the ultimate strength of the steel may not be reached (beam E). The apparent cause of failure is the crushing of the concrete.

Billet and Appleton studied both analytically and experimentally the behavior and ultimate flexural capacity of prestressed concrete beams [3].

Twenty six beams, 10 feet long, with a cross section of 6 in. x 12 in. were tested with equal loads at the third points of a 9 foot span. The beams were cast with two to fifteen 0.2 in. dia. wires in the tension zone. The high-strength wires extended in a straight line between two heavy steel plates at either end of the beam. A hole, 2 in. x 3 in. was centered 8 to 9 in. below the top of the beam. After the reinforcement was tensioned, the hole was grouted to complete the bond between the concrete and steel.

The beam tensile reinforcement was a high strength steel wire, cold-drawn, and stress-relieved or galvanized. The three types are stress relieved (Type I and II) and galvanized, used with the zinc coating removed (Type III).

The bond characteristics between the concrete and steel were improved by allowing the reinforcement to rust. The modulus of

elasticity of the wires, determined by strain measurements in tension tests, averaged 30,000,000 psi. The initial prestress stresses were 21,000 psi, 120,000 psi, and 160,000 psi. Some prestress was lost due to the relaxation of the steel and the creep of the concrete.

All beams were loaded until a compression failure of the concrete occurred or until one of the wires fractured after the concrete had been crushed.

The two modes of failure studied were the excessive elongation of the reinforcement ("under-reinforced") and the crushing of the concrete in the compression zone ("over-reinforced"). Both under- and over-reinforced beams reach maximum load when the concrete in the compression zone crushes. Billet and Appleton explain:

"At loads approaching failure of an under-reinforced beam, the reinforcement is stressed into the inelastic range. There is only a slight increase in load as the beam is deflected further. As a result of excessive elongation of the steel, cracks rise so that the compression zone is reduced until the internal compressive force can no longer be resisted by the concrete under the high strain gradient. Concrete generally crushes when extreme fiber strain reaches some limiting value ranging from about 0.0032 to 0.0040. Since an under-reinforced beam fails after steel enters the inelastic range, beam strength depends primarily on the strength of the steel. [3]"

"At loads approaching failure of an over-reinforced beam, cracks are lower than in an under-reinforced beam. As steel stress increases with further load, the resulting steel elongation allows cracks to progress higher. Near ultimate capacity of the beam, cracks cease to rise and in

some cases the neutral axis drops slightly. Finally a load is reached at which the capacity of the compression zone can no longer increase to balance increasing tensile force in the steel. At this stage flexural capacity is reached, and the compression zone crushes when the concrete reaches its limiting fiber strain. Since concrete crushes when stress in the steel is below yield stress, strength of an over-reinforced beam does not depend on tensile or yield strength of reinforcement, but primarily on the amount of reinforcement and strength of concrete. [3]"

Billet and Appleton assumed the following in their analysis of prestressed concrete:

- (1) Linear Strain Distributions.
- (2) Perfect bond between the concrete and reinforcement.
- (3) No tension resisted by the concrete.
- (4) Concrete crushes at ultimate strain.
- (5) Stress block of the concrete is defined by the three parameters k_1 , k_2 , and k_3 (ACI Code: β_1 , β_2 , β_3). Additional parameters are k_1k_3 and k_1d .
 - k_1 = ratio of average compressive stress to the maximum compressive stress of the concrete in the compression zone of the beam at ultimate.
 - k_2 = ratio of the depth to the line of action of the resultant compressive force in the compression zone to the depth to the neutral axis.
 - k_3 = ratio of maximum compressive strength of the concrete in flexure to the cylinder strength.
 - k_1k_3 = ratio of the average stress in the compression zone at failure to the cylinder strength.
 - k_1d = distance from extreme fiber in compression to neutral axis at ultimate.

(6) Stress-strain curve for steel is known.

By using conditions of equilibrium, Billet and Appleton [3] derived the following equations (see Figure 2.1) .

The ultimate moment can be defined as:

$$M_{ult} = T(d - k_2 k_u d) = f_{su} \rho b d^2 (1 - k_2 k_u) \quad (2.1)$$

where: M_{ult} = Ultimate Moment

f_{su} = Steel stress at maximum load

Dividing through by $k_1 k_3 f' c b d^2$, the equation reduces to:

$$\frac{M_{ult}}{k_1 k_3 f' c b d^2} = \frac{f_{su} \rho}{k_1 k_3 f' c} (1 - k_2 k_u) \quad (2.2)$$

The ratio k_u can be evaluated by equating the tensile force to the compressive force and dividing both sides by $k_1 k_3 f' c b d$.

$$k_u = \frac{f_{su} \rho}{k_1 k_3 f' c} \quad (2.3)$$

Substituting equation 2.3 into 2.2 yields:

$$\frac{M_{ult}}{k_1 k_3 f' c b d^2} = k_u (1 - k_2 k_u) \quad (2.4)$$

The ultimate moment of a prestressed beam can be calculated from the above equation. The only variable which is not known or cannot be assumed is k_u . The steel stress at failure, f_{su} , is also unknown.

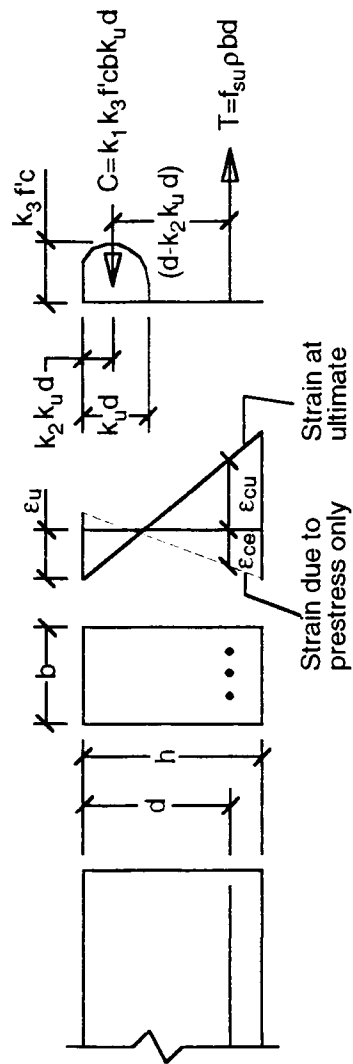


Figure 2.1 - Assumed strain and stress conditions

(Source: Billet, D. F., and Appleton, J. H., "Flexural Strength of Prestressed Concrete Beams," *ACI Journal*, June 1954, Proc. V. 50, p. 846.)

The variable k_u can be derived from strain compatibility and f_{su} can then be evaluated. Figure 2.1 shows the strain distribution which can be broken up into steps until ultimate is reached.

Stage I: The effect of prestress is considered. Strain in the concrete at the level of the steel is compressive. As load is applied, the concrete strain at the level of the steel approaches zero. At this load, the strain in the steel is equal to $\epsilon_{se} + \epsilon_{ce}$.

where: ϵ_{se} = steel strain due to effective prestress
 ϵ_{ce} = compressive strain in the concrete at the level of the steel due to the effective prestress force.

Stage II: The strain is at ultimate. Strain in the concrete is an ultimate strain ϵ_u . The strain in the concrete adjacent to the steel is ϵ_{cu} . A total strain in the steel at ultimate can then be defined by:

$$\epsilon_{su} = \epsilon_{se} + \epsilon_{ce} + \epsilon_{cu} \quad (2.5)$$

Relating ϵ_{cu} to ϵ_u and substituting, k_u can be solved with the following equation:

$$k_u = \frac{f_{su}\rho}{k_1 k_3 f'_c} = \frac{\epsilon_u}{\epsilon_{su} - \epsilon_{se} - \epsilon_{ce} + \epsilon_u} \quad (2.6)$$

For a beam, $\rho/k_1 k_3 f'_c$ and ϵ_u are known. ϵ_{se} and ϵ_{ce} can be calculated from the properties of the beam and reinforcement. The easiest and simplest way to determine the ultimate steel stress is through a graphical method. The graphical solution is obtained by plotting the following equation on the stress strain curve of the steel:

$$f_{su} = \frac{\epsilon_u E_s}{Q'(\epsilon_{su} - \epsilon_{se} + \epsilon_u - \epsilon_{ce})} \quad (2.7)$$

where: $Q' = E_s \rho / k_1 k_3 f'_c$.

$$\epsilon_{ce} \equiv [E_s \rho / f'_c] \times \epsilon_{se} \times 10^{-3}$$

The term $E_s \rho / k_1 k_3 f'_c$ is a convenient way of expressing the quantity $\rho / k_1 k_3 f'_c$. Figure 2.2 shows the intersection of the equation with the stress-strain curve of the steel yields the steel stress at ultimate.

The effect of variation of several parameters on the flexural capacity were observed. The parameter k_2 , which defines the location of the resultant of the internal compressive force, depends on the shape of the stress block (1/3 for triangular and 1/2 for a rectangular distribution). Billet and Appleton concluded that the effect of k_2 , when compared to the large variation of 1/3 to 1/2, had very little effect on the ultimate moment capacity; only those beams with high values of Q' were affected [3].

The ultimate moment capacity was very sensitive to the parameter $k_1 k_3$, which varied from 0.75 to 1.25. The value of $k_1 k_3$ was evaluated from test data for each beam tested. Test data indicated that concrete strength was the only quantity with which $k_1 k_3$ varied consistently.

The ultimate moment in the test beams proved to be in agreement with the calculated ultimate moment capacity. The largest deviations were for those beams with large values of Q' . The data

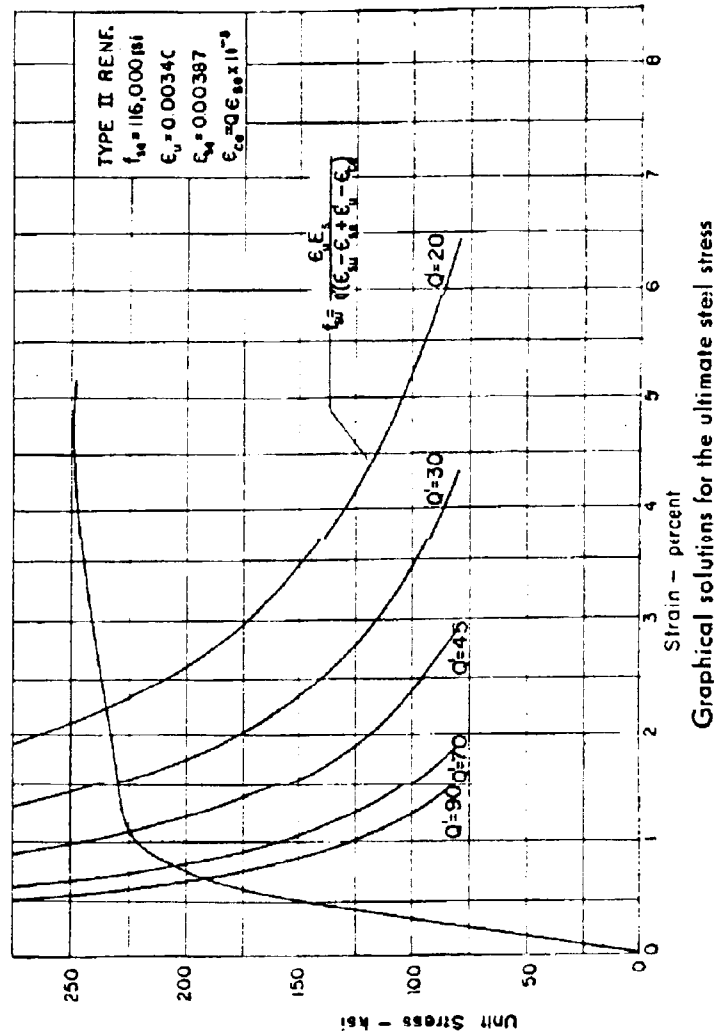


Figure 2.2 - Graphical solution for the ultimate steel stress

(Source: Billet, D. F., and Appleton, J. H., "Flexural Strength of Prestressed Concrete Beams," *ACI Journal*, June 1954, Proc. V. 50, p. 846.)

obtained is sensible because the concrete dictates the ultimate moment capacity for large values of Q' .

The following is a list of the major conclusions [3]:

- (1) The close agreement between analytical and experimental results indicates that the analytical expressions developed, derived from tests and containing empirical coefficients, will predict the ultimate flexural capacity of rectangular, bonded prestressed beams.
- (2) Both test results and analyses indicate that the effective tensioning stress has no real effect on the ultimate flexural capacity of under-reinforced beams. Increasing the prestress in over-reinforced beams, however, increases the flexural capacity.
- (3) The magnitude of Q' has a large effect on the ultimate deflection. As Q' is increased, the deflection at the ultimate load is decreased.
- (4) The modulus of rupture is an adequate value for the ultimate tensile stress.

Janney et al [11] examined the flexural behavior of prestressed beams. Three different steel ratios were used to determine moment-deflection relationships, deflection recovery, and the ultimate strength in flexure. Analytical results were compared to experimental results.

Prestressed beams with a cross-section of 6"x12" were tested. Beams were 9 feet long (clear span) and were loaded at the third points. The strength of the concrete was approximately 5500 psi in all tests.

Steel tendons were 3/8" seven-wire strand (nominal area = 0.08 sq. in) with an effective prestress of 120 ksi. The strands used were rusted intentionally to obtain a good bond to the concrete. Steel ratios varied from 0.322 to 0.965 percent. The strands were not stress-relieved which provided an earlier than expected leave from the elastic range (see Figure 2.3).

All beams were tested to failure. In all the beams, the stress in the prestress strands at ultimate load ranged over the upper half of the stress-strain curve. Fracture of the steel occurred in only one beam.

The analysis developed by Janney et al for the ultimate flexural capacity was based on the work done by Stüssi [11,17]. Two sets of equations, statics and strain compatibility, were involved.

Figure 2.4 shows the following force equilibrium condition:

$$T = f_{su}As = C = k_1k_3f'cbc \quad (2.8)$$

where: c = depth of compression zone at M_{ult}
 f_{su} = stress in reinforcement at M_{ult}
 d = effective depth (8.3 in) of steel

Eliminating the depth to the neutral axis, c , and defining the empirical relationship $k_u = c/d = q_u/k_1k_3$, equation 2.9 reduces to:

$$\frac{M_{ult}}{f'cbd^2} = q_u \left(1 - \frac{k_2}{k_1k_3} q_u \right) \quad (2.10)$$

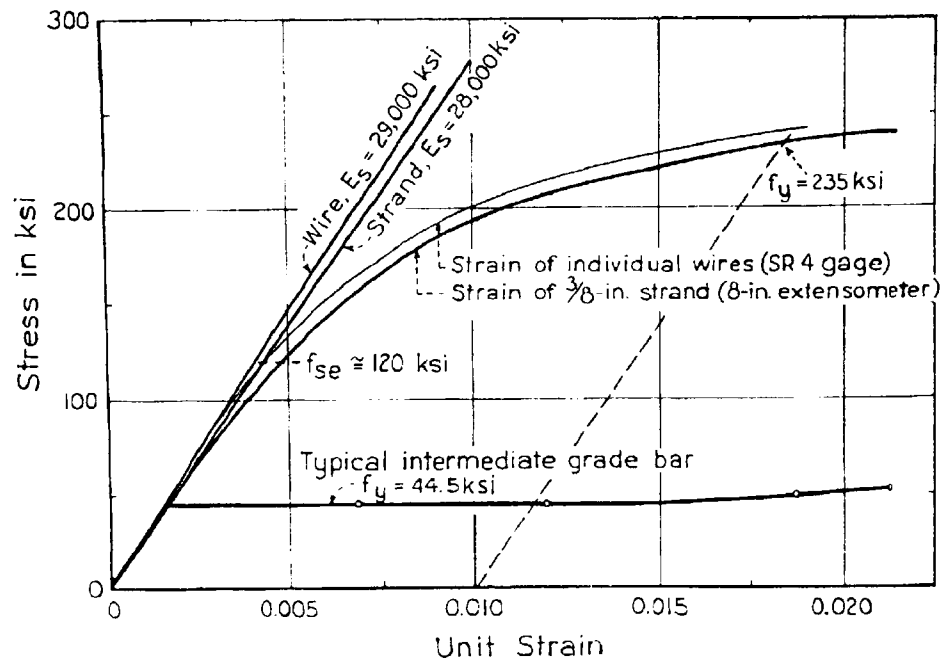


Figure 2.3 - Stress-strain curves for reinforcement

(Source: Janney, J.R., Hognestad, E. and McHenry, D., "Ultimate Flexural Strength of Prestressed and Conventionally Reinforced Concrete Beams," *ACI Journal* Vol. 52, Jan. 1956, p. 606)

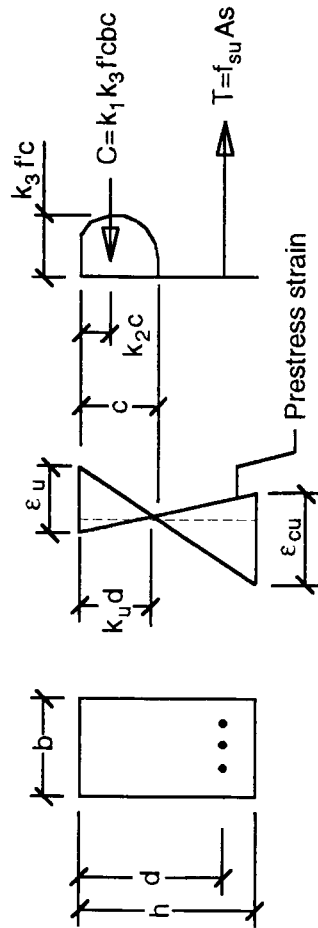


Figure 2.4 - Assumed strain and stress conditions

(Source: Janney, J.R., Hognestad, E. and McHenry, D., "Ultimate Flexural Strength of Prestressed and Conventionally Reinforced Concrete Beams," *ACI Journal* Vol. 52 Jan. 1956, p. 612)

$$\text{where: } q_u = \rho \frac{f_{su}}{f'_c} \quad (2.11)$$

k_2 = coefficient determining position of the internal compressive force
Billet and Appleton assumed $k_2 = 0.42$. For the studies of ultimate flexural capacity, $k_2/k_1k_3 = .52$ and $k_1k_3 = .83$ and are reported in the results.

The strain compatibility is defined as:

$$\epsilon_{cu} = \frac{\epsilon_u(1 - k_u)}{k_u} \quad (2.12)$$

where: ϵ_u = ultimate strain in concrete
 ϵ_{cu} = strain in concrete at level of steel at ultimate

Baker suggested that the following relationship could be determined experimentally [2]:

$$F = \frac{\Delta_{\text{steel-strain}}}{\Delta_{\text{concrete-strain}}} \quad (2.13)$$

$F = 1.00$ for bonded tendons

The total strain ϵ_{su} in the prestressed steel at ultimate is the sum of the effective prestress ϵ_{se} and the change in strain of the concrete at the level of the steel times F .

$$\epsilon_{su} = \epsilon_{se} + F\epsilon_{cu} \quad (2.14)$$

Substituting equation 2.13 into 2.14 yields:

$$\epsilon_{su} = \epsilon_{se} + \frac{F\epsilon_u}{k_u} - F\epsilon_u \quad (2.15)$$

Setting $c = k_u d$, equations 2.8 and 2.15 yield:

$$F \epsilon_u = \frac{(\epsilon_{su} - \epsilon_{se}) q_u}{k_1 k_3 - q_u} \quad (2.16)$$

The effective prestress steel strain minus the change in steel strain due to loading to failure ($\epsilon_{su} - \epsilon_{se}$) was measured from the tests [11]. Based on the stress-strain curve, $F \epsilon_u$ was computed from equation 2.16 assuming $k_1 k_3 = 0.83$. Maximum concrete strains, ϵ_u , varied from 0.0025 to 0.0040 and the corresponding value of F varied from 1.1 to 0.7. The calculated ultimate moments were based on $F = 1.0$ and $\epsilon_u = .003$ and are tabulated in Table 2.2. Figure 2.5 shows the relationship of measured ultimate moment and measured steel stress at failure.

In conclusion, the ultimate strength analysis, based on the Stüssi principle, permitted prediction of the variables which affect ultimate flexural behavior [11,17].

Table 2.2 - Properties of Beams and Test Results

Beam - qy	Steel	Percent	f'c (psi)	f _{se} (ks)	Mult (test) (K-in)	Mult (calc) (K-in)	Mtest Mcalc
1-0.141	2 strands	0.322	5350	119	318	292	1.09
1-0.250	4 strands	0.644	6050	113	547	493	1.11
1.0.420	6 strands	0.965	5400	117	580	615	0.94

(Source: Janney, J.R., Hognestad, E. and McHenry, D., "Ultimate Flexural Strength of Prestressed and Conventionally Reinforced Concrete Beams," *ACI Journal* Vol. 52, Jan. 1956, p. 604)

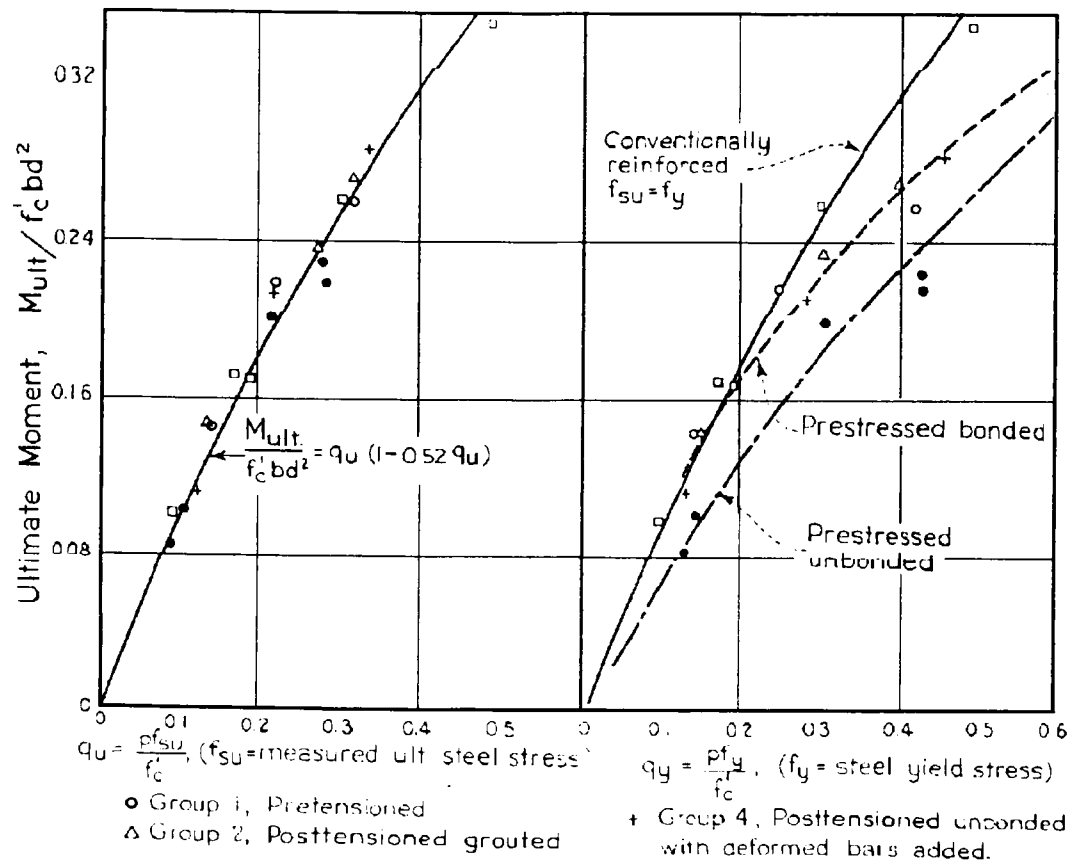


Figure 2.5 - Relation between measured M_{ult} and measured q_u

(Source: Janney, J.R., Hognestad, E. and McHenry, D., "Ultimate Flexural Strength of Prestressed and Conventionally Reinforced Concrete Beams," *ACI Journal* Vol. 52 Jan. 1956, p. 612)

Chapter III

Test Program

The AASHTO Type-I prestressed concrete bridge beams which were tested at The University of Tennessee at Knoxville (UTK) were modeled with the finite element program ABAQUS. The program was used to determine whether the load-deflection behavior and the ultimate strain conditions of the prestressed beams could be predicted via a finite element program. The test results from ABAQUS were compared with the experimental results [5], and subsequent correlations were made.

Beam Designation

Because the initial load-deflection plots are compared to the actual test data, the designation of the beams is that of the beams tested at the University of Tennessee at Knoxville with a slight variation. A three part designation was used to designate each beam [5]. To that designation, an initial prestress value and an ultimate strain are added. The five divisions of the designation are as follows:

Division I

- 5 = 1/2" Regular strand, nominal diameter of 0.5000"
- 5S = 1/2" Special strand, nominal diameter of 0.5224"
- 916 = 9/16" strand, nominal diameter of 0.5625"
- 6 = 6/10" strand, nominal diameter of 0.6000"

Division II

The second part refers to the number of beams prestressed with the strands. For the beams from the strand manufacturer's group, the second part refers to the name of the manufacturer.

Division III

INT - interior end of the beam
EXT - exterior end of the beam
EAST - east end of beam

Division IV

180 - fse = 180 ksi
185 - fse = 185 ksi
190 - fse = 190 ksi
195 - fse = 195 ksi
200 - fse = 200 ksi (etc.)

Division V

This is the value of the ultimate compressive strain used in the finite element analysis (i.e. .003 represents an ultimate compressive strain = .003).

The ABAQUS Model

The beam which was modeled was a slight variation of the AASHTO Type-I beam. This variation was due primarily to geometric and user element complications. Figure 3.1 shows the properties of the AAHSTO Type-I beam. Figure 3.2 and 3.3 show the properties and the Nodal Layout of the revised AASHTO Type-I beam respectively (noted

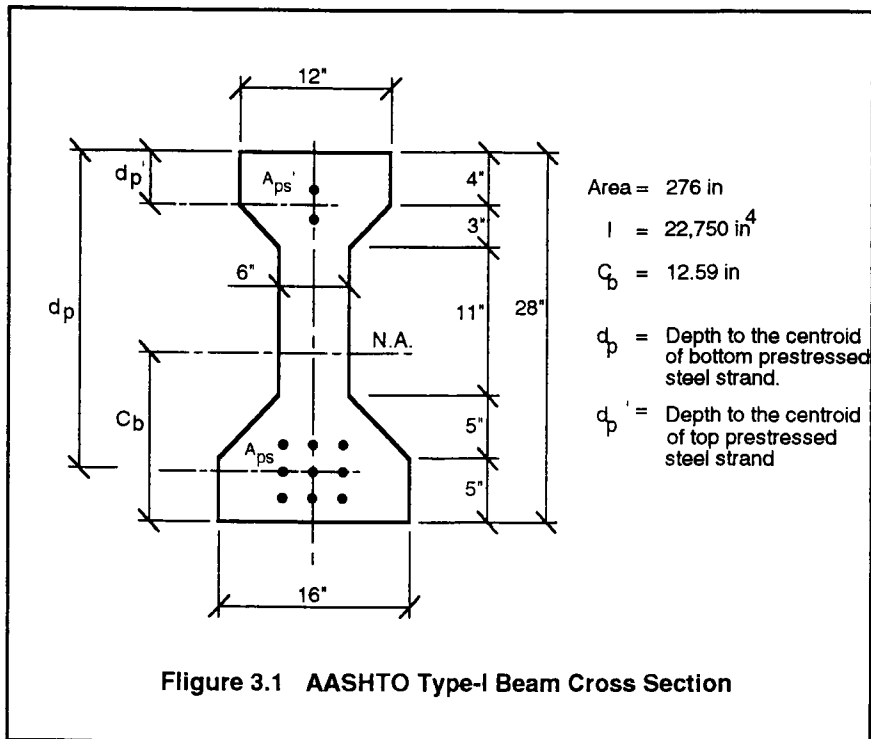


Figure 3.1 AASHTO Type-I Beam Cross Section

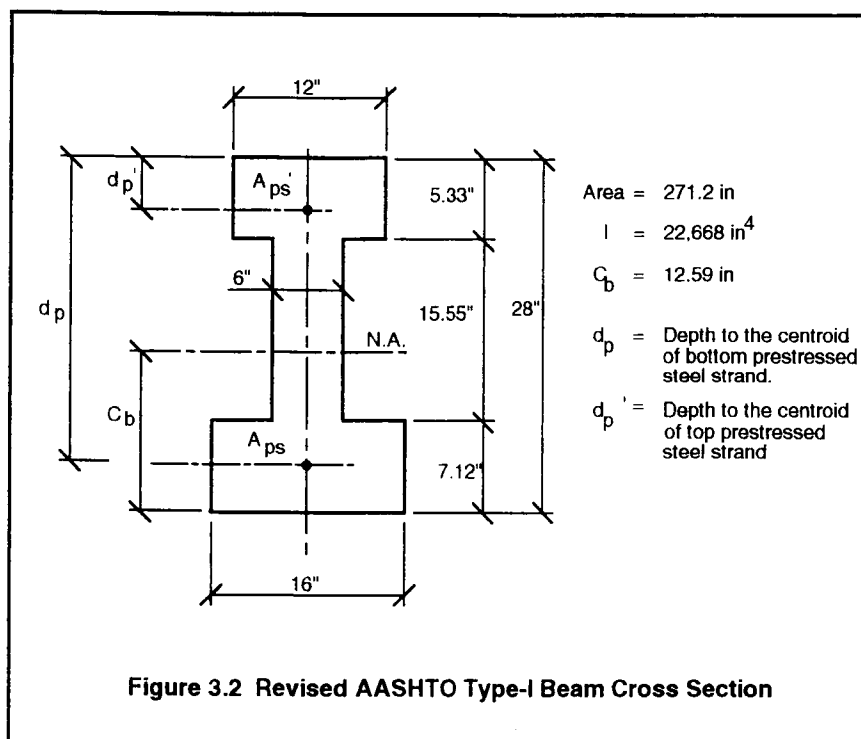


Figure 3.2 Revised AASHTO Type-I Beam Cross Section

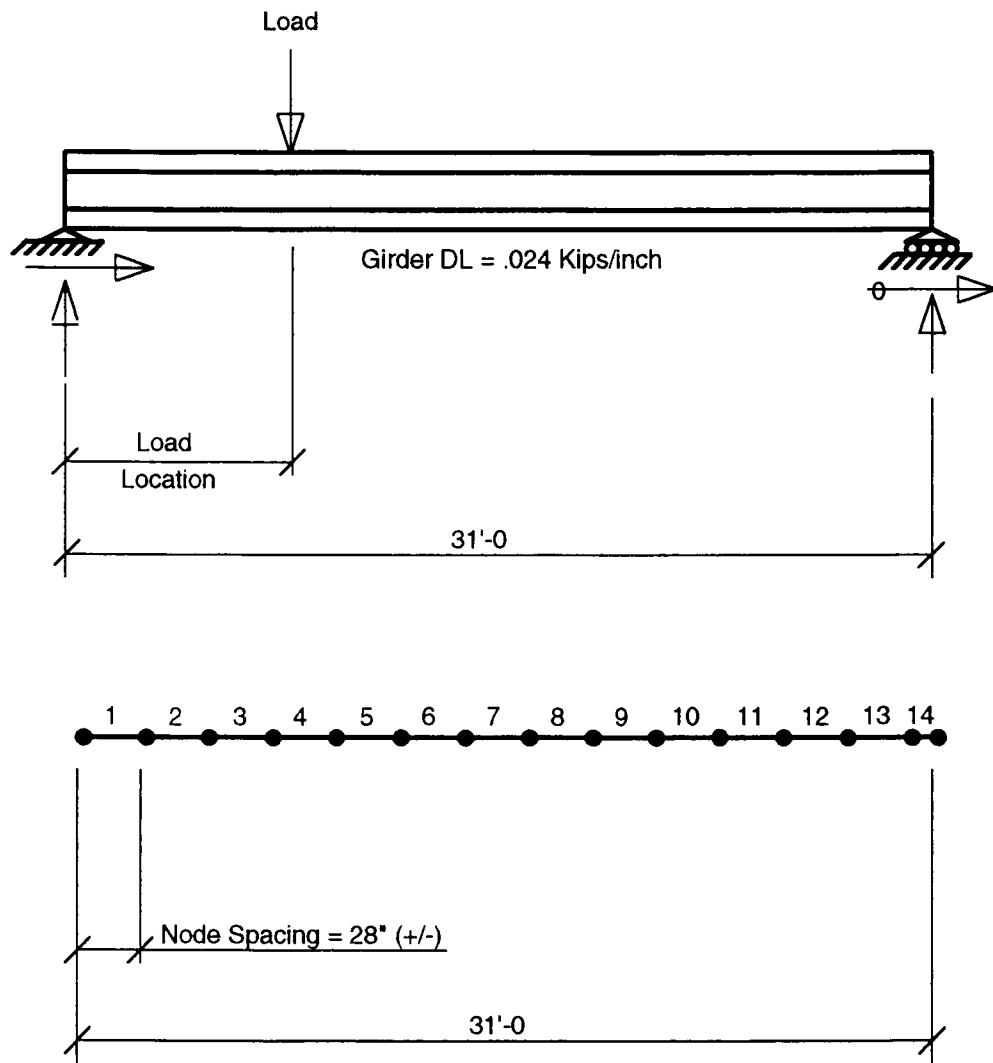


Figure 3.3 Nodal Layout of Revised AASHTO Type-I Beam

RAASHTO Type-I). All of the RAASHTO Type-I beams in the analysis were 31 feet long and 28 inches deep. The beam was broken into 14 elements which were 28" (+/-) long. This distance was chosen in order to set the height to length ratio of the elements to approximately one. The elasto-plastic properties for both the concrete and steel were then defined (see ABAQUS Concrete Model and High Strength Steel Modeling). Prestress was applied to the beam and the beam came to equilibrium under the dead load of the beam. An external load (see Figure 3.3) was applied to the beam and incremental load-deflection, concrete stress, concrete strain, steel stress and steel strain data were obtained. This incremental data defined the elasto-plastic behavior of the RAASHTO Type-I beam. The analysis stopped when the ultimate strain in the concrete was reached.

Node and Element Designation

The optimum element length for analyzing the behavior at ultimate was determined to be 28" (+/-) only after many variations of element lengths were tested. It was also determined from the extensive testing of element variation that a small change in the element length produced substantial variations in the results at ultimate, while a change in the height to length ratio of one or two of the elements near the load location had little effect on the cracking behavior of the beam. It was for this reason that the height to length ratio of the elements was set to unity for results at ultimate and variations of the height to length ratio were

allowed to predict the cracking behavior of the beam. The ultimate stresses and strains were calculated at the integration point which was approximately 14" away from the load point ($1/2$ times the element length). The element length for the cracking model was also set to 28"(+/-). In order to predict the cracking behavior, the element length adjacent to the load was set to one inch.

The element chosen for the analysis was the Abaqus B21 [8] beam element. This element is a linear beam element which uses Timoshenko beam theory. It is assumed that the beam cross-section is plane initially to the beam axis and that plane sections remain plane (warping is not considered). The beam element B21 has one integration point and includes transverse shear deformations. Because the Abaqus beam element considers bending and axial strains, a smoother response and better results were obtained as compared to the three-dimensional elements. It is noted that the three-dimensional elements did not "bend" and therefore the beam exhibited a very "stiff" behavior.

The cross section shown in figure 3.2 was used because of the I-shaped beam section for beam elements available from Abaqus [8]. The I-Section element (Simpson Integration) provided an easy yet effective way to model the AASHTO Type-I beam. For beams in a plane, the I-shaped cross section allows output to be at 5 points (at the flanges plus three points in the web).

The ABAQUS Concrete Model

The reinforced concrete model in ABAQUS was used to define the properties of plain concrete outside the elastic range [8]. It is assumed that the concrete will be coupled with rebar elements which will be used to model the reinforcing [9]. The rebar elements are superimposed on a mesh of plain concrete so that the behavior of the concrete will be independent from the steel. Bond, slip, and dowel action cannot be modeled; however the load transfer across cracks through the rebar can be modeled using tension stiffening. In order to describe the non-linear behavior of concrete, ABAQUS uses the two material property definitions *ELASTIC and *CONCRETE.

The option *ELASTIC [8] is used to define the linear modulus of elasticity. The simplest form of *ELASTIC requires the variables of the modulus and poisson's ratio and assumes an isotropic, homogeneous material. The chord-modulus was used to describe Young's modulus for the *ELASTIC option in the analysis. Tests [13] have indicated that the static-elastic modulus (Young's modulus) can be described best by the chord-modulus method (ASTM C469). The chord-modulus of elasticity is defined as the slope of the chord of the uniaxial stress-strain curve between the point corresponding to 40 percent of maximum stress and the point corresponding to a strain of 0.00005. The chord modulus is calculated from the relation:

$$E_c = \frac{S_2 - S_1}{\epsilon_2 - 0.00005} \quad (3.1)$$

where S_2 is the stress corresponding to 40 percent of the Ultimate stress, S_1 is the stress corresponding to a strain of .00005, and ϵ_2 is the longitudinal strain corresponding to S_2 . The longitudinal strain was set to a value of .0005 for the model.

Figure 3.4 shows the idealized stress-strain curve for concrete in uniaxial compression [13,15]. The concrete is initially loaded in compression at zero stress and zero strain. The concrete then exhibits an elastic response in the elastic range up until a stress of 0.40 f'c and a strain of .0005 (+/-). As the stress is increased up to a maximum stress of f'c (strain = .002), some non-recoverable inelastic strain occurs and the material softens until it cannot carry any additional stress. The concrete then loses strength until it reaches 85% of the maximum stress.

Poisson's Ratio (the ratio between the transverse strain and the strain in the direction of the applied uniaxial loading) was set to 0.18 for the model [15].

The material definition *CONCRETE [8] is used with several options (*FAILURE RATIO, SHEAR RETENTION, and *TENSION STIFFENING) and is used to describe both the behavior and compressive stress-strain relationship of concrete in the inelastic range. The modeling of concrete in ABAQUS is designed for relatively monotonic loadings under low confining pressures [9]. The confining pressures are limited to four to five times the largest compressive stress that can be carried in uniaxial compression. Cracking and post-cracking behavior are the most important attributes of the model. The cracking occurs when "crack

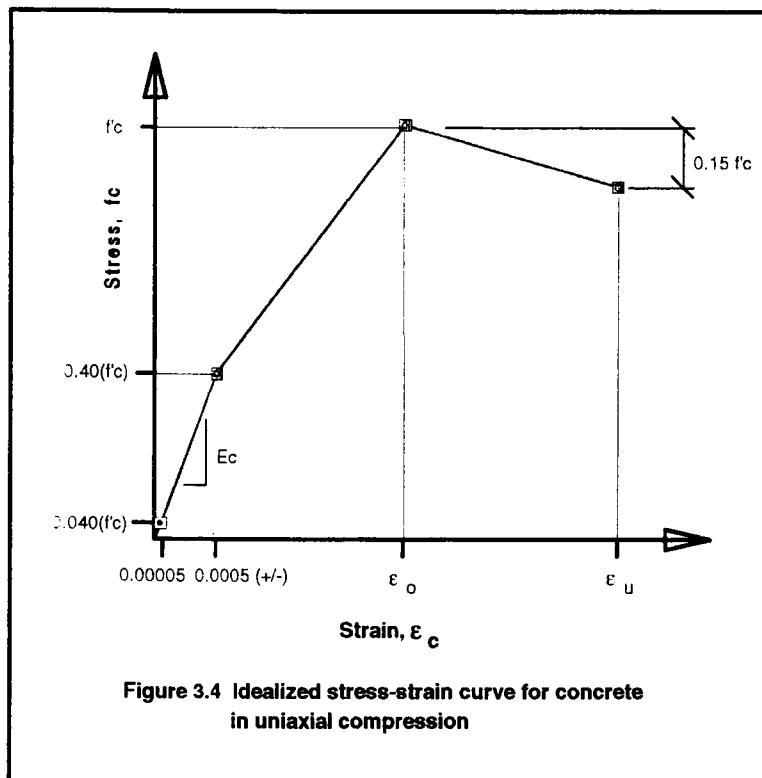
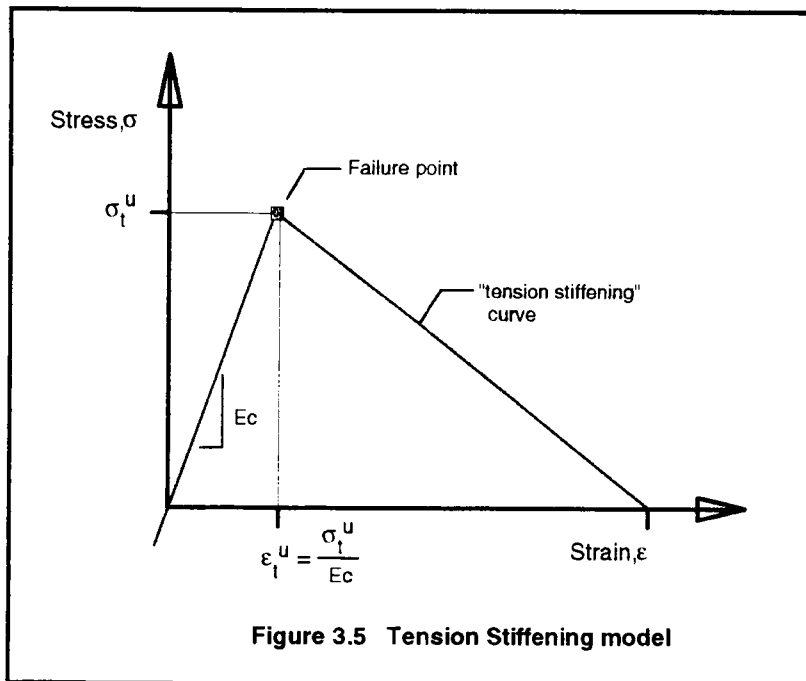


Figure 3.4 Idealized stress-strain curve for concrete in uniaxial compression

detection surface" reaches an ultimate tensile stress. The cracked model then exhibits a strain softening behavior which is described by the *TENSION STIFFENING [8] option (see Figure 3.5).

The Tension Stiffening option describes both the fraction of the remaining stress to the stress at cracking and the absolute value of the tensile strain minus the failure strain at cracking. No additional tensile stress was assumed after cracking and the change in strain of the concrete was set to the effective prestress value divided by the Modulus of Elasticity for the prestress for the AASHTO Type-I beam. This assumption was based on linear strain distributions, plane section remain plane, and the tensile failure strain is small with respect to the tensile strain of the prestress. Because the crack detection surface depends on the tensile stress of the concrete and not the strain, the ultimate tensile stress (or cracking stress) is described by the *FAILURE RATIO [8] option. The *FAILURE RATIO option has two parameters, the ratio of the ultimate biaxial stress to the uniaxial compressive stress, and the absolute value of the ratio of the ultimate uniaxial tensile stress to the ultimate uniaxial compressive stress.

It has been accepted by the American Concrete Institute (ACI) that a 0.5 power relationship exists between the tensile strength and the compressive strength of concrete. However, recent research [13] indicates that the tensile strength is proportional to the 0.79 power. For $f'_c > 1000$ psi and for any age concrete, the tensile strength of concrete



can be computed from the relationship:

$$f_t = .584 [f'_c]^{.79} \quad (3.2)$$

The relationship above was used in the analysis.

High Strength Steel Modeling

The high strength prestressing steel modeled was a 7-wire, low relaxation strand with an ultimate tensile strength of 270 ksi. Data indicate that the 7-wire strand (Florida Wire and Cable Co.) actually has an ultimate stress of 275 ksi (see Figure 3.6).

Table 3.1 shows the area and the relative depths of the steel for beams 5-3-EXT and 5-SWAI-EAST.

Table 3.1 - Depths and Areas of Steel.

5-3-EXT				5-SWAI-EAST			
Aps	Aps'	dp	dp'	Aps	Aps'	dp	dp'
1.224	.306	24.25	4.0	1.224	.306	24.47	4.0

Abaqus defines steel through a *REBAR option [8]. This option is used to define rebars in beams as well as continuum elements. The elasto-plastic behavior rebar is then defined through it's own material property definition, different from the material definition of the surrounding elements. Prestress is applied and held constant with the

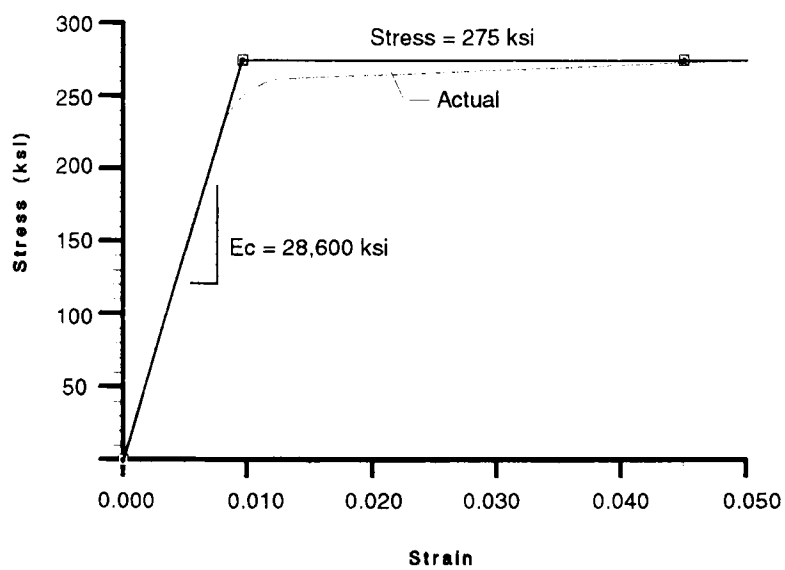


Figure 3.6 Stress-strain curve for 270 ksi, 7-wire strand (Low Relaxation)

*PRESTRESS HOLD [8] option, and the beam is allowed to come to equilibrium under its own dead load.

The *PRESTRESS HOLD allows the full prestress to be applied to the entire beam. In other words, the transfer length, or the length of bar required to transfer the steel stress into the concrete, is equal to zero. It would not be reasonable to assume that any load applied to the beam inside the development length would yield acceptable results for ultimate behavior.

Result Generation

Abaqus uses a history dependent model which uses a series of small increments to develop a solution. The two main factors in a history dependent model are the solution of the discrete equilibrium statement and how the increment size is chosen. Abaqus uses Newton's method for the solution of the equation.

A static analysis is defined through the option *STATIC [8] and allows the user to specify a time increment and a total time for the solution. The static analysis assumes that all inertia forces are zero or at most vary slowly in the model. Because a nonlinear static equilibrium solution to an elasto-plastic problem was sought, the modified Riks algorithm was used. The Riks algorithm is used to detect load-displacement response where the load or displacement may decrease

as the time-step increments increase. The Riks algorithm also assumes proportional loading, or all load magnitudes vary with a single scalar parameter, and that the response will be reasonably smooth [9].

A static point load was applied. Both concrete and steel stresses and strains were evaluated at the integration points. Also, load-deflection data was obtained at the load point.

Chapter IV

Discussion of Results

Results

The final results of the bridge beams tested are listed in Table 4.1, 4.2, and 4.3. Figure 4.1 shows a typical load-deflection plot generated by ABAQUS. Also shown in the figure are the variables evaluated at cracking, nominal, and at rupture. The variables are:

1. Cracking - M_{cr} , kd_{cr} , & ϕ_{cr}
2. Nominal - M_n , kd_n (kd_n tabulated for beam 5-SWAI-EAST in table 4.3)
3. Rupture - f_{bs} , f_{ts} , kd_r , & ϕ_r

The nominal moment was evaluated at the ultimate concrete compressive stress while the curvature at rupture was evaluated at the ultimate or rupture strain of the concrete.

In order to understand the effects of variables on the moment-curvature behavior of the AASHTO beam, the following interaction between the concrete and steel is assumed:

1. The tendons are fully bonded to the concrete. A change in strain in the steel is equal to a change in strain of the concrete.
2. The stress-strain properties of the steel and concrete are known.
3. Strains are linear and are assumed to be distributed over the depth of the beam.
4. Tension and compression forces are equal.
5. The failure analyzed is a flexural failure. There is no bond, slip, or shear failure.

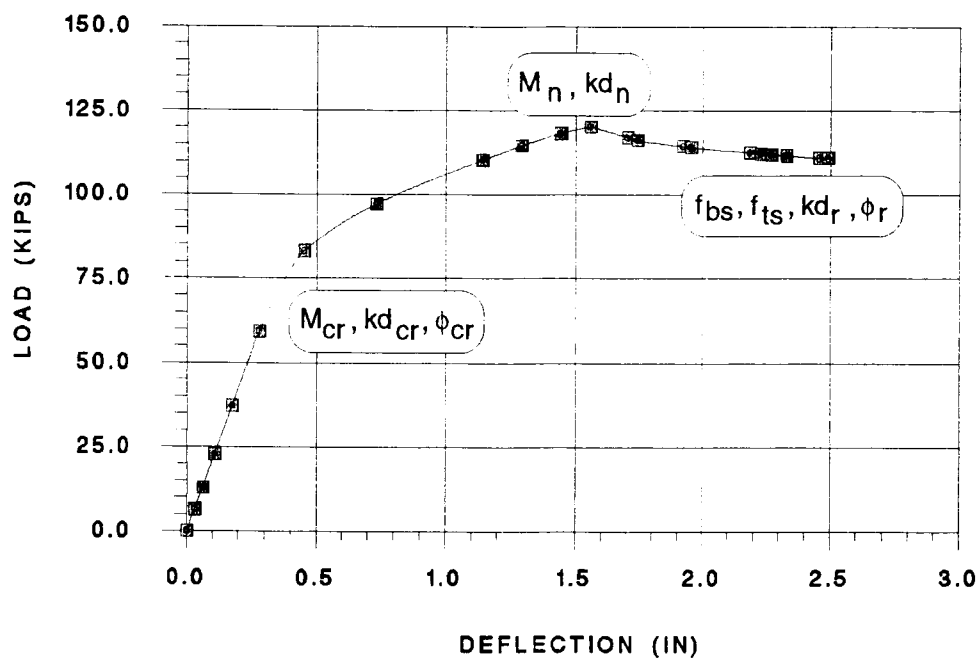


Figure 4.1 - Typical Variable Evaluation Location on a Typical Load-Deflection Plot

Table 4.1 Beam 5-SWAI-EAST Analysis ($f'_c = 5.55$ ksi, ϵ_c varies, f_{se} varies)

ϵ_{conc} (in/in)	σ_{conc} (ksi)	f_{se} (ksi)	M_{cr} (kip-ft)	ϕ_{cr} (in ⁻¹)	kd_{cr} (in)	M_n (kip-ft)	ϕ_r (in ⁻¹)	kd_r (in)	f_{hs} (ksi)	ϵ_{bs}	f_{fs} (ksi)
.003	5.55	190	433.60	2.86E-5	24.07	682.0	38.84E-5	7.72	275	13.15E-3	122.30
.004	5.55	190	433.60	2.86E-5	24.07	682.0	57.29E-5	6.98	275	16.67E-3	99.38
.005	5.55	190	433.60	2.86E-5	24.07	682.0	73.52E-5	6.80	275	19.63E-3	79.99
.006	5.55	190	433.60	2.86E-5	24.07	682.0	90.85E-5	6.60	275	22.87E-3	58.22
.003	5.55	195	440.00	2.88E-5	24.27	681.49	37.00E-5	8.10	275	12.88E-3	127.3
.004	5.55	195	440.00	2.88E-5	24.27	681.49	56.70E-5	7.05	275	16.69E-3	104.7
.005	5.55	195	440.00	2.88E-5	24.27	681.49	72.90E-5	6.85	275	19.67E-3	85.26
.006	5.55	195	440.00	2.88E-5	24.27	681.49	90.45E-5	6.63	275	22.96E-3	62.05
.003	5.55	200	445.14	2.90E-5	24.34	683.10	37.2E-5	8.05	275	13.11E-3	131.20
.004	5.55	200	445.14	2.90E-5	24.34	683.10	55.36E-5	7.22	275	16.54E-3	111.00
.005	5.55	200	445.14	2.90E-5	24.34	683.10	72.40E-5	6.90	275	19.70E-3	90.37
.006	5.55	200	445.14	2.90E-5	24.34	683.10	90.06E-5	6.66	275	23.03E-3	68.30

ϵ_{conc} = Concrete strain at rupture

M_{cr} = Cracking moment

ϕ_{cr} = Curvature at cracking

kd_{cr} = kd at cracking

σ_{conc} = Concrete stress at nominal moment

M_n = Nominal moment capacity

ϕ_r = Curvature at concrete rupture strain

kd_r = kd at concrete rupture strain

f_{se} = Effective prestress

f_{bs} = Bottom steel stress at rupture

ϵ_{bs} = Bottom steel strain at rupture

f_{fs} = Top steel stress at rupture

Table 4.2 Beam 5-3-EXT Analysis ($f'_c = 6.86$ ksi, ϵ : varies, f_{se} : varies)

ϵ_{conc} (in/in)	σ_{conc} (ksi)	f_{se} (ksi)	M_{cr} (kip-ft)	ϕ_{cr} (in ⁻¹)	kd_{cr} (in)	M_n (kip-ft)	ϕ_r (in ⁻¹)	kd_r (in)	f_{hs} (ksi)	ϵ_{bs}	f_{fs} (ksi)
.003	6.86	150	376.5	2.19E-5	23.44	682.38	48.27E-5	6.21	275	13.95E-3	86.53
.004	6.86	150	376.5	2.19E-5	23.44	682.38	66.36E-5	6.00	275	17.39E-3	62.96
.005	6.86	150	376.5	2.19E-5	23.44	682.38	82.36E-5	6.08	275	20.20E-3	43.09
.006	6.86	150	376.5	2.19E-5	23.44	682.38	97.1E-5	5.93	275	23.80E-3	23.22
.003	6.86	191	458.50	2.48E-5	23.70	685.66	46.27E-5	6.48	275	14.90E-3	126.50
.004	6.86	191	458.50	2.48E-5	23.70	685.66	63.54E-5	6.29	275	18.09E-3	105.50
.005	6.86	191	458.50	2.48E-5	23.70	685.66	81.04E-5	6.16	275	21.33E-3	83.32
.006	6.86	191	458.50	2.48E-5	23.70	685.66	98.62E-5	6.08	275	24.59E-3	60.94
.003	6.86	195	460.80	2.55E-5	23.60	685.11	46.20E-5	6.49	275	15.02E-3	130.40
.004	6.86	195	460.80	2.55E-5	23.60	685.11	63.54E-5	6.30	275	18.23E-3	109.30
.005	6.86	195	460.80	2.55E-5	23.60	685.11	80.23E-5	6.23	275	21.27E-3	88.48
.005	6.86	195	460.80	2.55E-5	23.60	685.11	97.66E-5	6.14	275	24.50E-3	66.08
.003	6.86	200	461.96	2.60E-5	23.87	684.56	46.04E-5	6.52	275	15.10E-3	135.50
.004	6.86	200	461.96	2.60E-5	23.87	684.56	63.65E-5	6.28	275	18.43E-3	113.60
.005	6.86	200	461.96	2.60E-5	23.87	684.56	80.35E-5	6.22	275	21.48E-3	92.56
.006	6.86	200	461.96	2.60E-5	23.87	684.56	97.15E-5	6.17	275	24.55E-3	71.54

ϵ_{conc} = Concrete strain at rupture

M_{cr} = Cracking moment

ϕ_{cr} = Curvature at cracking

kd_{cr} = kd at cracking

σ_{conc} = Concrete stress at nominal moment

M_n = Nominal moment capacity

ϕ_r = Curvature at concrete rupture strain

kd_r = kd at concrete rupture strain

f_{se} = Effective prestress

f_{bs} = Bottom steel stress at rupture

ϵ_{bs} = Bottom steel strain at rupture

f_{st} = Top steel stress at rupture

Table 4.3 Beam 5-SWAI-EAST Analysis ($f'_c = 5.55$ ksi, $f'_c = 15$ ksi, $\epsilon = .003$)

ϵ_{conc} modeled	ϵ_{conc} @ M_n	σ_{conc} modeled	σ_{conc} @ M_n	f_{se} (ksi)	M_{cr} (kip-ft)	ϕ_{cr} (in^{-1})	kd_{cr} (in)	M_n (kip-ft)	kd_n (in)
.003	.001938	5.55	5.55	190	433.60	2.86E-5	24.07	682.00	9.96
.003	.001966	5.55	5.55	200	445.14	2.90E-5	24.34	683.10	10.07
.003	.000754	15.0	7.52	190	669.52	1.83E-5	22.74	824.60	7.45
.003	.000765	15.0	7.59	200	681.64	1.88E-5	22.88	843.06	7.66

ϵ_{conc} = Concrete strain at rupture
 M_{cr} = Cracking moment
 kd_{cr} = kd at cracking
 σ_{conc} = Concrete stress at nominal moment
 M_n = Nominal moment capacity
 kd_n = kd at nominal moment
 f_{se} = Effective prestress
 ϕ_{cr} = Curvature at cracking

Effects of Effective Prestress

The effective prestress of the steel was varied in each beam to different degrees.

Table 4.2 shows that for a 33 percent increase in the effective prestress, there was a 23 percent increase in the cracking moment. The increase in the effective prestress increased both the compressive stress and strain in the bottom fibers of the concrete at time $t=0$ to time $t=\text{crack}$. After the "cracking state", the bottom fibers in the concrete exhibited a strain softening behavior until the stress was equal to zero .

Tables 4.1 and 4.2 indicate relatively small changes in the nominal moment and curvature at rupture with changes in the effective prestress. The change in the nominal moment is small because the steel has been stressed into the inelastic range (ultimate stress) and the compression zone has been reduced by cracking. High strain gradients rise and the concrete crushes at maximum concrete strain. Billet and Appleton [3] describe this behavior as "under-reinforced". Figures 4.1 and 4.2 show the relationship between the effective prestress (f_{se}) and k_d . The graph indicates that a small change in the effective prestress produces a relatively small change in k_d and therefore small changes in the nominal moment.

Effects of Ultimate Concrete Strain

Table 4.1 and 4.2 show that for a given concrete stress and effective prestress, a change in the ultimate compressive strain in the concrete produces a change in the curvature (deflection) and the steel strain at rupture. The curvature at cracking, the cracking moment, and the nominal moment, however, are not affected by the ultimate compressive strain in the concrete.

The ultimate concrete strain is plotted against k_d at ultimate for Beam 5-3-EXT and Beam 5-SWAI-EAST in Figures 4.2 and 4.3 respectively. The curve is best described by a polynomial fit. For small strains, k_d increases asymptotically to a k_d defined by effective prestress plus the dead load of the member. As the ultimate concrete strain increases, k_d decreases asymptotically to a value governed by the ultimate steel strain. Park and Paulay suggest an ultimate steel strain of approximately 5% [15].

Effects of Ultimate Concrete Stress

Table 4.3 indicates that for a given steel ratio, an increase in the ultimate concrete stress produces an increase in the cracking moment and the nominal moment. The increase in ultimate concrete stress, however, decreases the rotation of the beam at cracking and thereby decreasing k_{dCr} . The increase in the ultimate concrete stress, f'_c , causes a direct increase in the modulus of elasticity and an increase in the tensile capacity of the concrete. Because the modulus of elasticity

Ultimate Concrete Strain vs. k_d for Beam 5-3-EXT

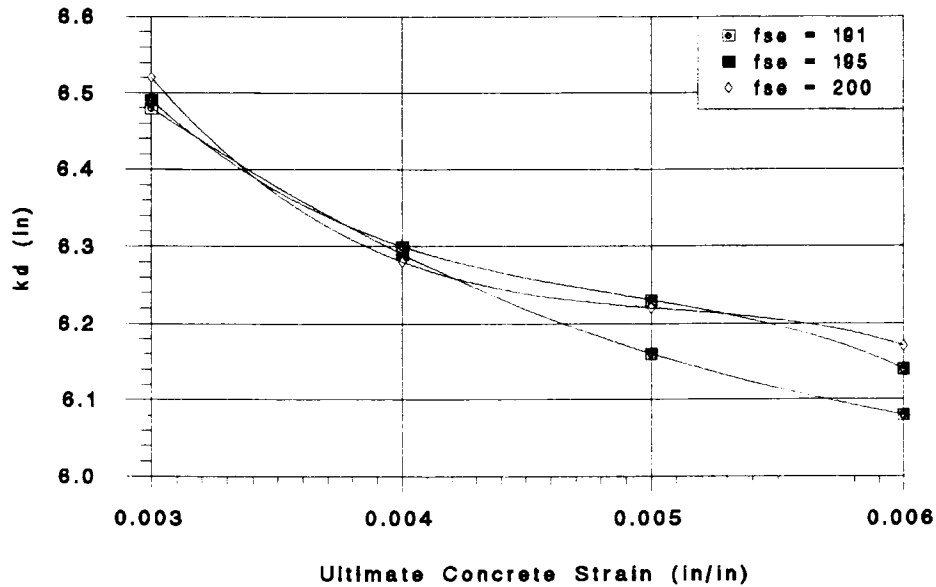


Figure 4.2 - Ultimate Concrete Strain vs. k_d for Beam 5-3-EXT

Ultimate Concrete Strain vs. k_d for Beam 5-SWAI-EAST

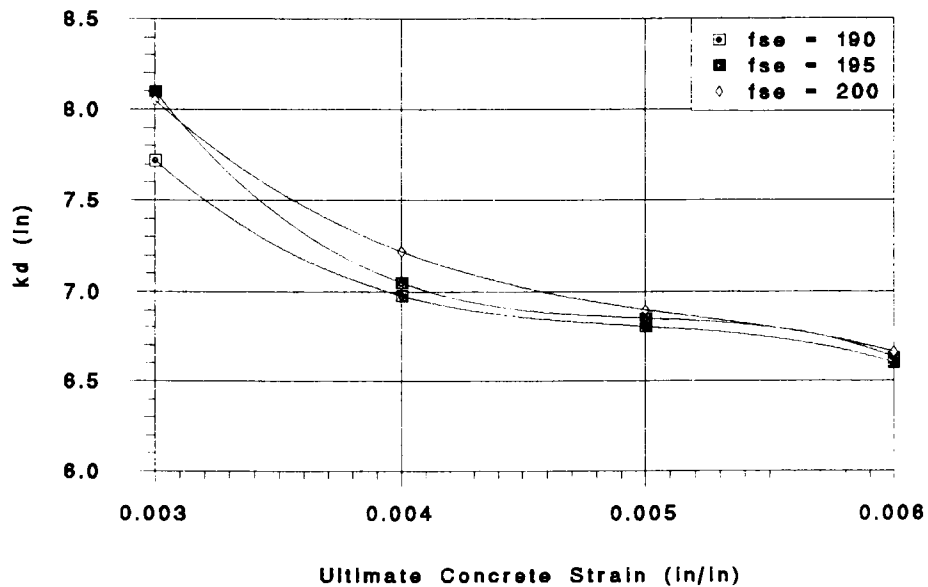


Figure 4.3 - Ultimate Concrete Strain vs. k_d for Beam 5-SWAI-EAST

increases with 15.0 ksi concrete, the compressive strain decreases at cracking. The curvature of the 15 ksi beams decrease with a decrease in k_d because the compressive strain decreases at a higher rate than $k_{d_{cr}}$ decreases.

There was a greater change in the nominal moment for the 15.0 ksi high strength concrete tests. Table 4.3 indicates a 20 percent increase in the nominal moment with an approximately 40 percent increase in strength. Closer examination of the computed results, however, shows a large discrepancy between the computed and theoretical nominal moments. Even though the concrete compressive force was approximately equal to the steel tension force, the summation of moments about the bottom steel yielded moment capacities much less than those computed from the external loads. An explanation of the errors in the high strength concrete model lies in Abaqus' smeared crack model. The smeared crack model does not track individual "macro" cracks; instead, calculations are performed by the way the cracks effect the stress and material stiffness associated with the integration point [9]. The primary objection to the smeared crack model is that the modeling approach inherently introduces mesh sensitivity in the solution [9].

Abaqus Results vs. University of Tennessee's Results

Beams 5-3-EXT and 5-SWAI-EAST were used to determine if the adequate results could be determined via a finite elements program (ABAQUS). The two beams which were chosen from the UTK study were loaded at the minimum development length (i.e. that length of prestressing steel needed to develop the full moment capacity of the beam). The development length of each beam depends on the strand diameter as well as the prestress and the effective prestress. Only beams which failed in flexural compression and did not exhibit any prestress loss due to slip from bond failure were considered. Table 4.4 shows the ABAQUS and the UTK results for beams 5-SWAI-EAST and 5-3-EXT. Ratios between the UTK and ABAQUS results were calculated to ascertain the performance of computer simulated models versus laboratory tests. A coefficient of 1.00 defines perfect correlation.

Recall that ABAQUS defines a cracked section when the concrete tensile stress of the member is reached and not when the stress approaches zero (i.e after the tension stiffening). The cracking moment results are based on the ultimate tensile stress of the concrete. Strain softening is not considered in the computation of the cracking moment. The UTK and ABAQUS results are compared to a computed cracking moment based on the ultimate tensile stress:

$$f_t = .584 [f'_c]^{.79} \quad (4.1)$$

Table 4.4 - University of Tennessee Results & ABAQUS Results

Condition	UTK Results		ABAQUS Results		ABAQUS/UTK	
	5-SWAI-EAST	5-3-EXT	5-SWAI-EAST-200-003	5-3-EXT-191-004	5-SWAI-EAST	5-3-EXT
f'c (ksi)	5.55	6.86	5.55	6.86	1.00	1.00
Load Location (in)	81	85.1	81	85.1	1.00	1.00
Mcr (kip-ft)	430	388	445	458	1.03	1.20
Mcr (computed)	437	431	437	431	1.00	1.00
Mcr/Mcr (computed)	.99	.90	1.02	1.06	1.02	1.18
Mn (kip-ft)	641	675	683	686	1.07	1.02
Mode of Failure	Flex. Comp.	Flex. Comp.	Flex. Comp.	Flex. Comp.	Flex. Comp.	Flex. Comp.
Mn/Mcr	1.49	1.75	1.54	1.49	1.03	0.85
Mn/Mnc	1.05	1.10	1.12	1.13	1.07	1.03
fse (ksi)	200	191	200	191	1.00	1.00
fps (bottom steel) (ksi)	-----	-----	275	275	-----	-----
εps (bottom steel) (in/in)	-----	-----	13.11 E-3	18.09 E-3	-----	-----
fps (calculated) (ksi)	-----	-----	275	275	-----	-----
εps (calculated) (in/in)	-----	-----	12.98 E-3	20.05 E-3	-----	-----
εps (abaqus)/εps (calculated)	-----	-----	1.01	.90	-----	-----

(Calculated ACI nominal moments, calculated steel stresses, and calculated steel strains are based on $\epsilon_{conc} = .003$)

Mnc = From strain compatibility and statics

Mnc(ACI)(5-3-EXT)=607 K-FT

Mnc(ACI)(5-SWAI-EAST)=611 K-FT

Based on the experimental results, when compared to the computed, the above equation yielded cracking moments within ten percent.

The computed nominal moment capacity based on the ACI Code yielded substantially lower results than either the UTK or ABAQUS tests. Results varied from 5% to 13%. The ratio between ABAQUS and UTK, however, varied between 2% and 7% for beams 5-3-EXT-191-004 and 5-SWAI-EAST-200-003 respectively.

The ACI Code, section 18.8.3, requires the total amount of prestressed and nonprestressed reinforcement of flexural members to be adequate to develop a design moment strength at least equal to 1.2 times the cracking moment strength ($\phi M_n \geq 1.2 M_{cr}$). The provision exists so that flexural failure will not occur due to the rupture of the prestressing tendons after cracking. When comparing the results to the ACI Code, it is important to note that the ACI cracking moment is based on elastic theory using a modulus of rupture equal to:

$$f_r = 7.5\sqrt{f'_c} \quad (4.2)$$

Both the UTK and ABAQUS tests yielded $\phi M_n/M_{cr}$ ratios from 1.3 to 1.58. The weakest ratio between the UTK and ABAQUS results was the M_n/M_{cr} ratio equal to .85. This weak correlation is primarily due to the unusually low cracking moment of Beam 5-3-EXT of the UTK tests.

Ultimate steel stresses and equivalent strains for the bottom prestressing strands were reached for both beams in the ABAQUS tests as well as in the ACI Code calculations. A 1% to 10% correlation exists between ABAQUS bottom steel strains and calculated steel strains.

ABAQUS vs. UTK load -deflection data are plotted for Beam 5-SWAI-EAST and Beam 5-3-EXT and are shown in Figures 4.4 and 4.5 respectively. It can be seen from the plots that the chord modulus (E_c) offers a good representation of the actual elasticity of concrete. The plots also indicate (esp. Fig. 4.5) that a maximum usable strain can be evaluated with good reliability using ABAQUS.

Load vs. Deflection for Beam 5-SWAI-EAST
 (ABAQUS RESULTS - BEAM 5-SWAI-EAST-200-003)

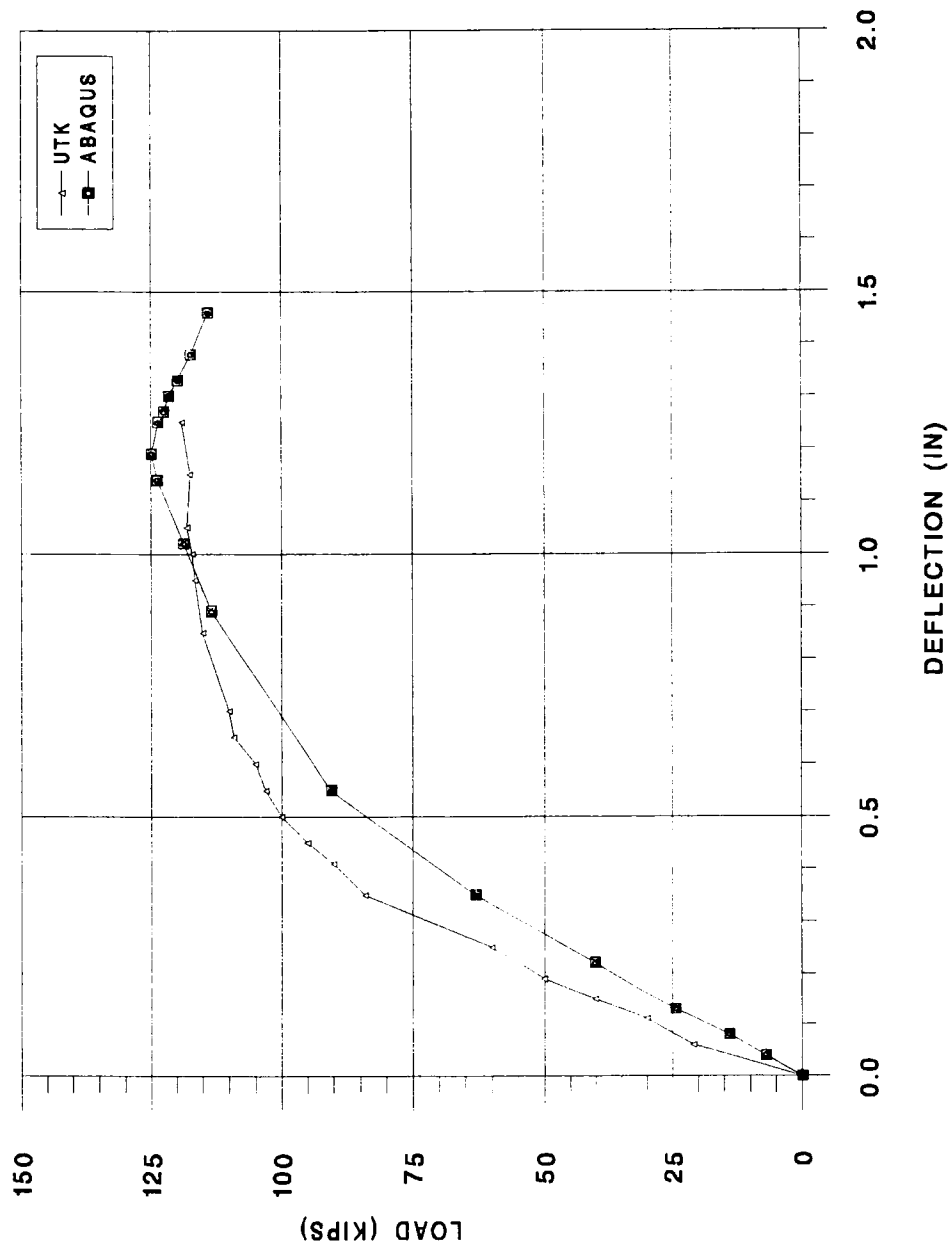


Figure 4.4 - Load - Deflection Plot for Beam 5-SWAI-EAST

Load vs. Deflection for Beam 5-3-EXT
 (ABAQUS RESULTS - BEAM 5-3-EXT-181-004)

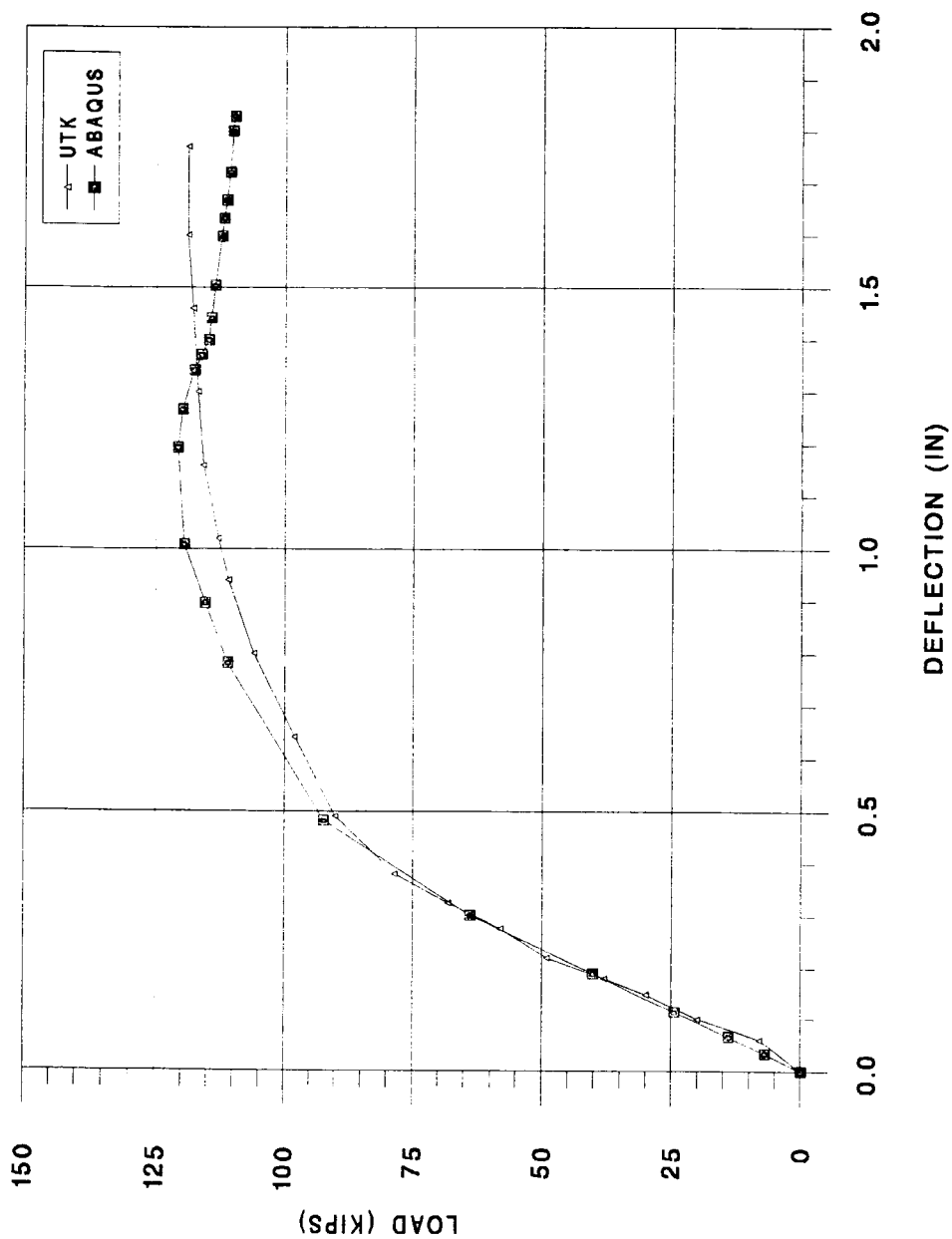


Figure 4.5 - Load - Deflection Plot for Beam 5-3-EXT

Chapter V

Conclusions

A computer simulated analysis on AASHTO Type-I beams was performed using the finite elements program, ABAQUS. The cracking and inelastic behavior were studied and compared to both experimental and analytical results. The conclusions of the ABAQUS tests are as follows:

The computer simulated analysis when compared to the experimental and analytical results indicate that the finite elements program, ABAQUS, will satisfactorily predict both the cracking capacity and the nominal flexural capacity of an AASHTO Type-I beam.

The effective prestress of the steel generally had little effect on the nominal moment capacity for the AASHTO Type-I beams.

The change in the ultimate concrete strain in the under-reinforced beams produced a noted change in the ultimate curvature (deflection) of the beam and the ultimate steel strain but no significant change was observed in the nominal moment capacity.

Load-deflection plots indicate that the ultimate concrete strain evaluated from Abaqus is within reason with experimental knowledge of ultimate concrete strain.

The tests performed on the high-strength concrete did not perform satisfactorily. One possible explanation was numerical error due to mesh sensitivity in the smeared crack model.

LIST OF REFERENCES

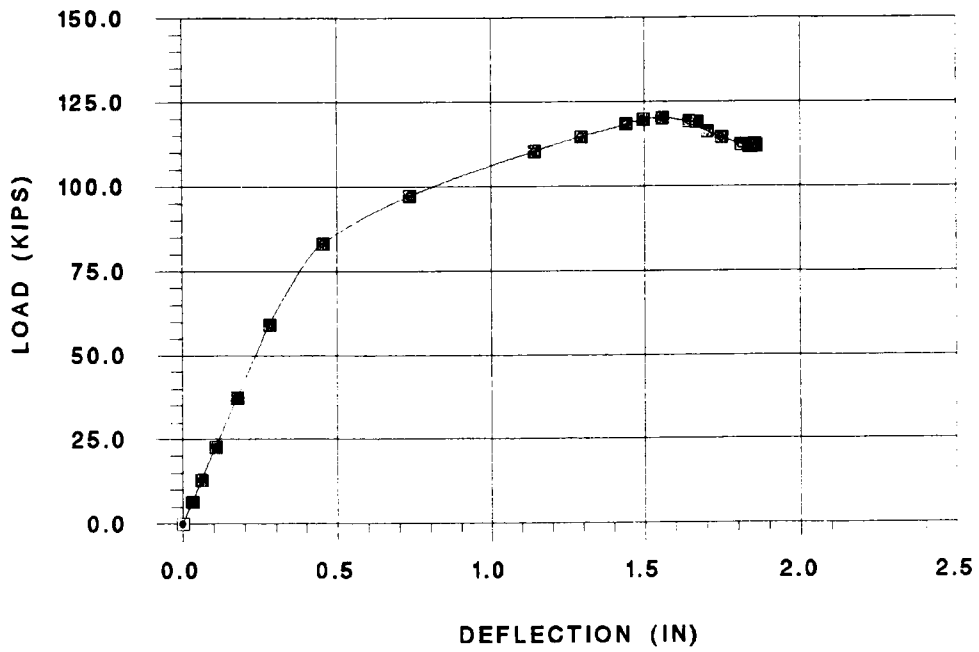
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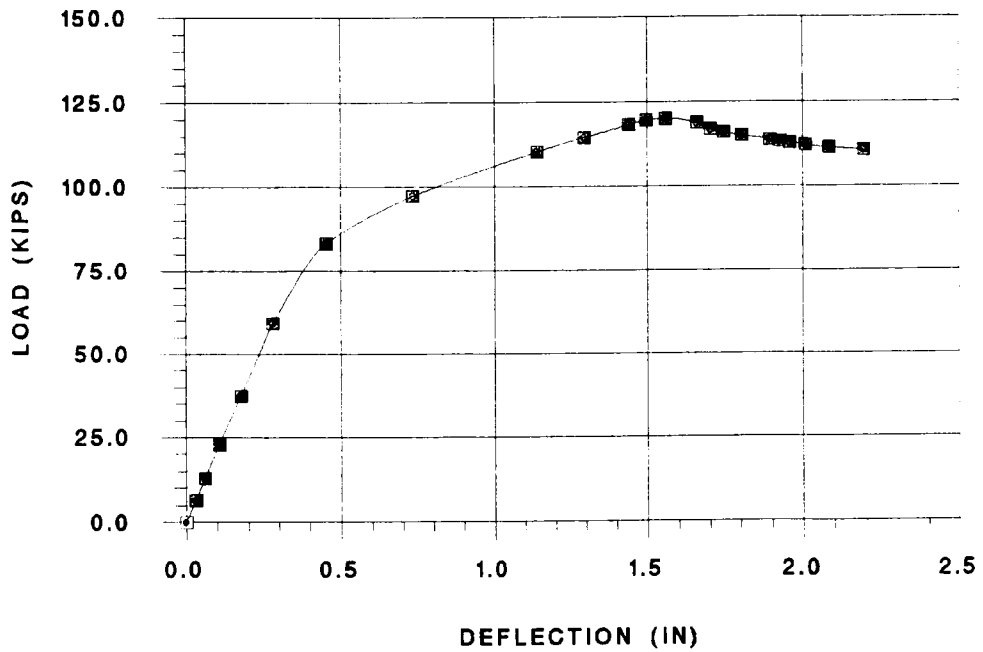
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APPENDIX

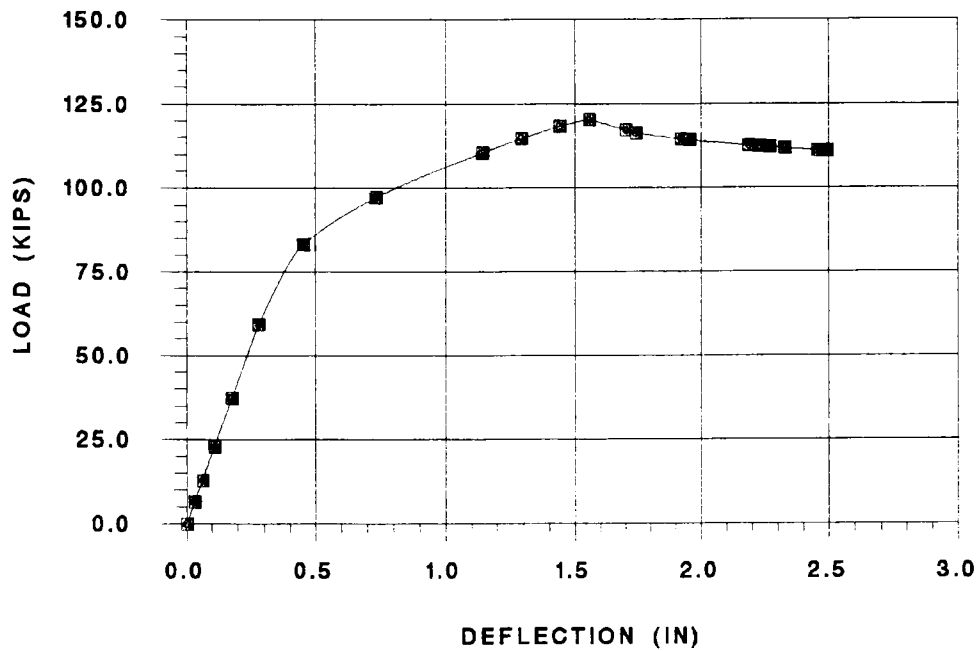
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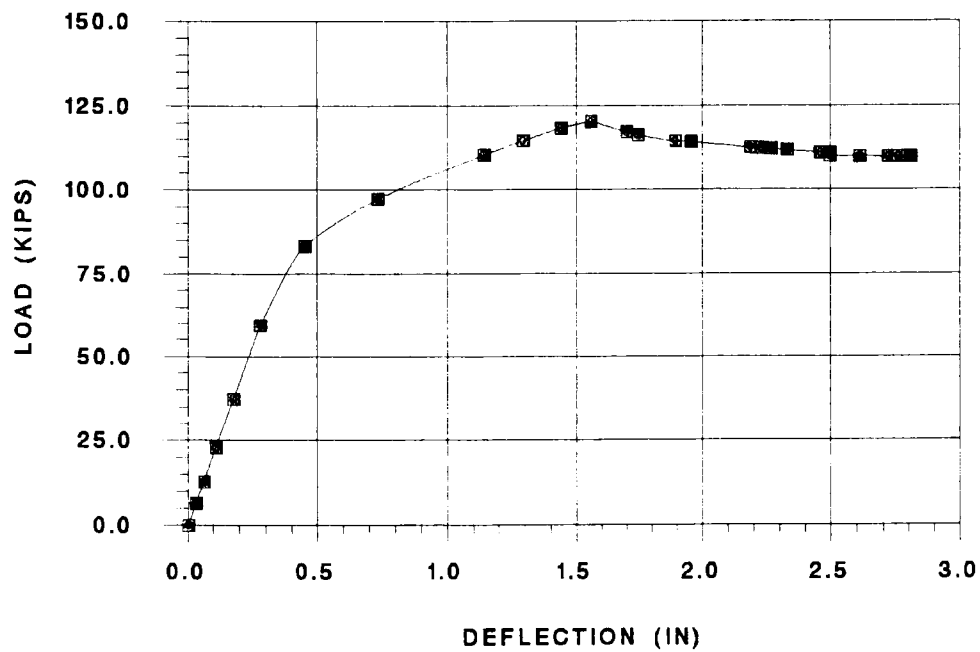
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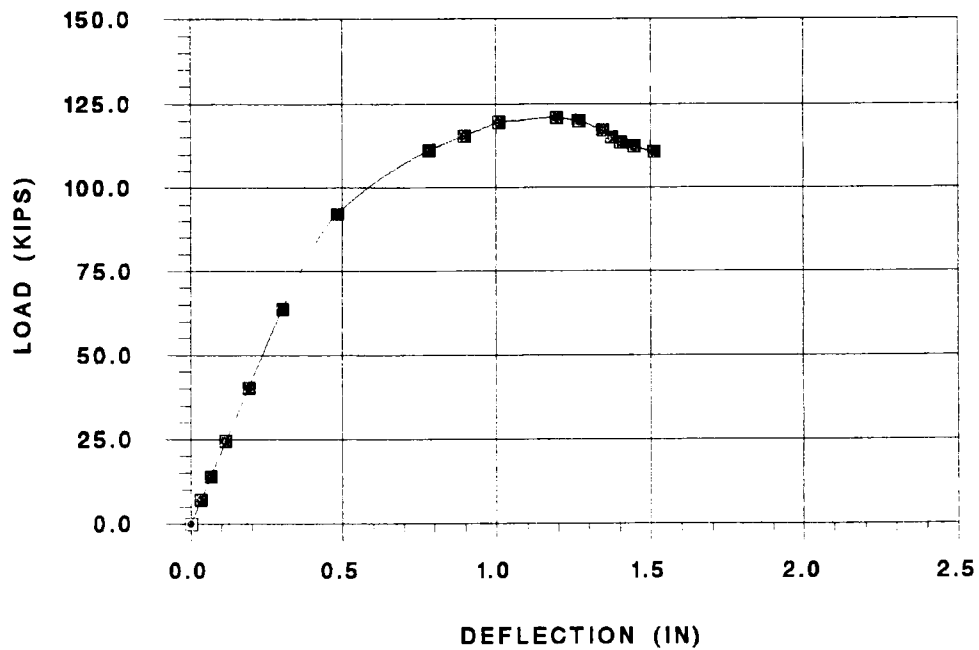
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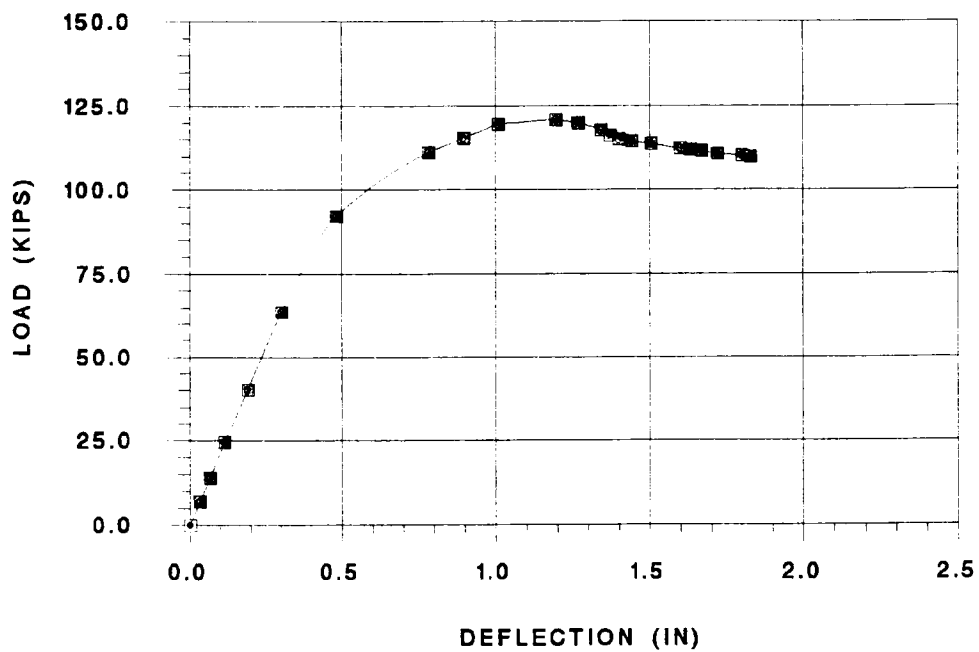
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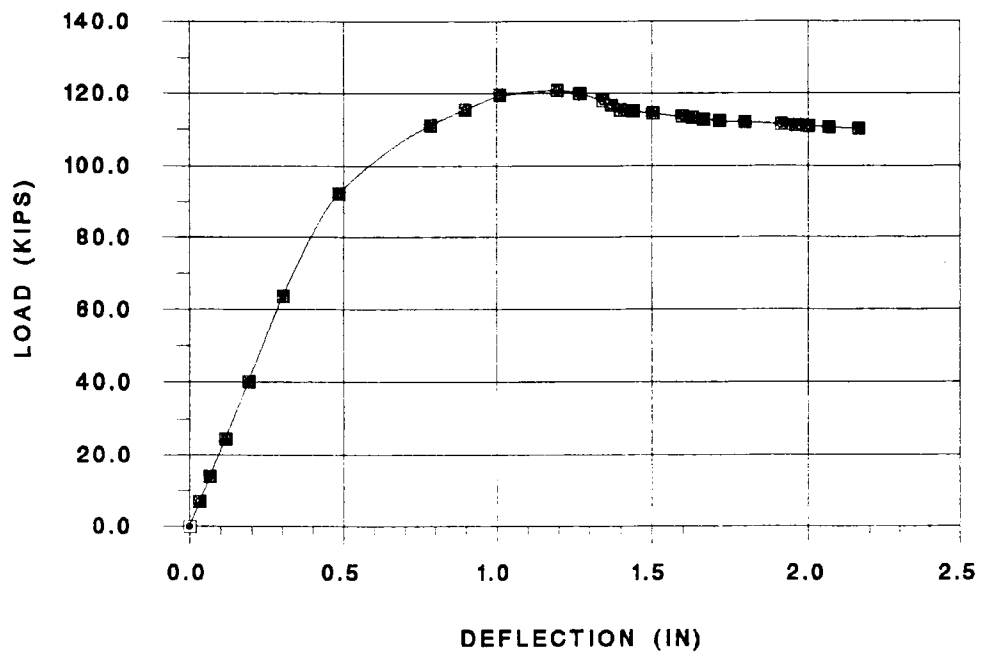
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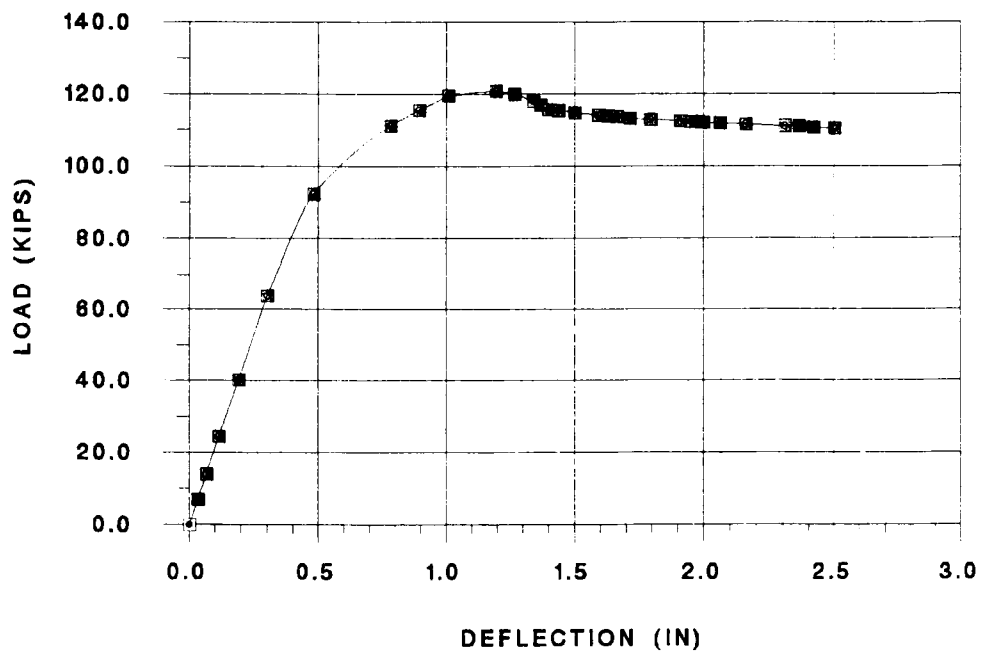
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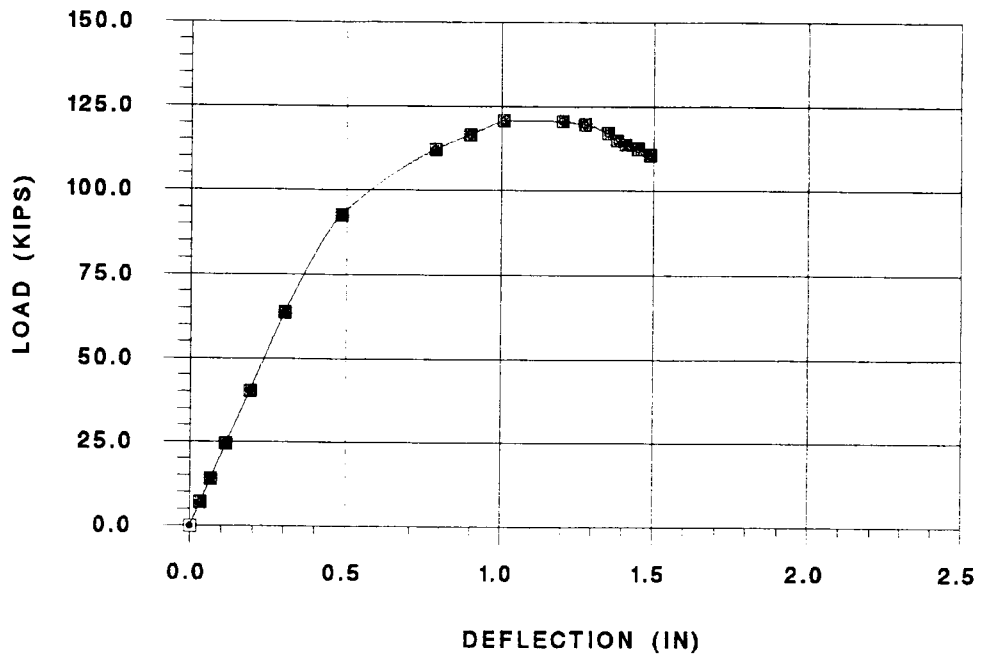
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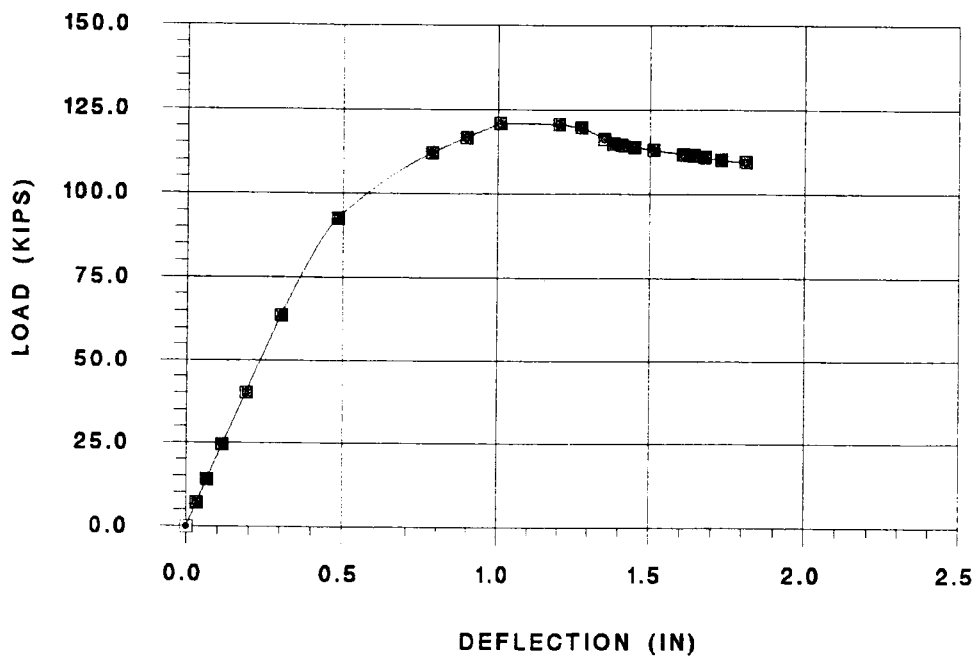
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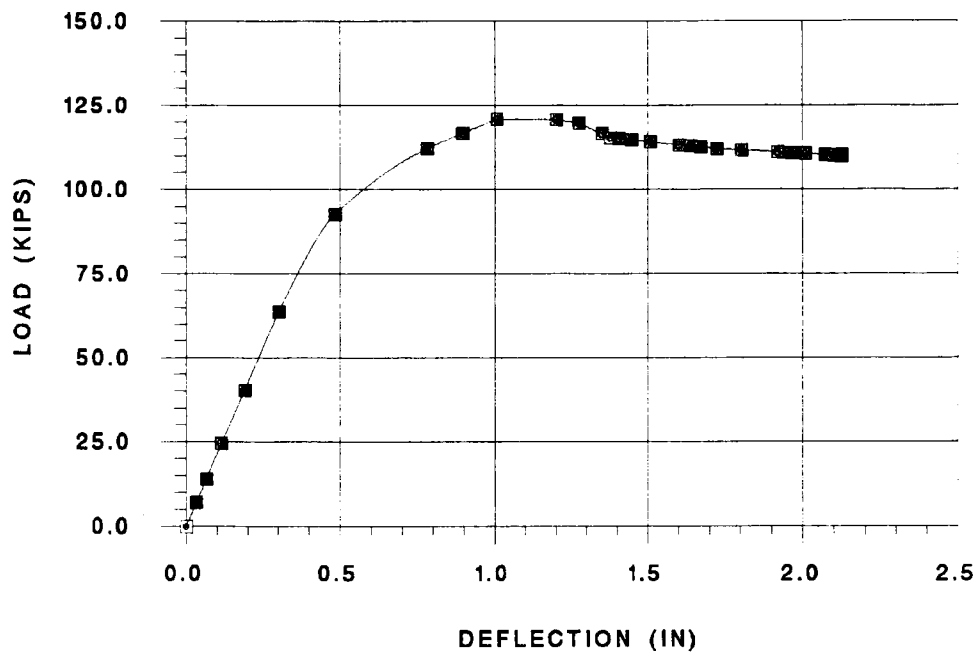
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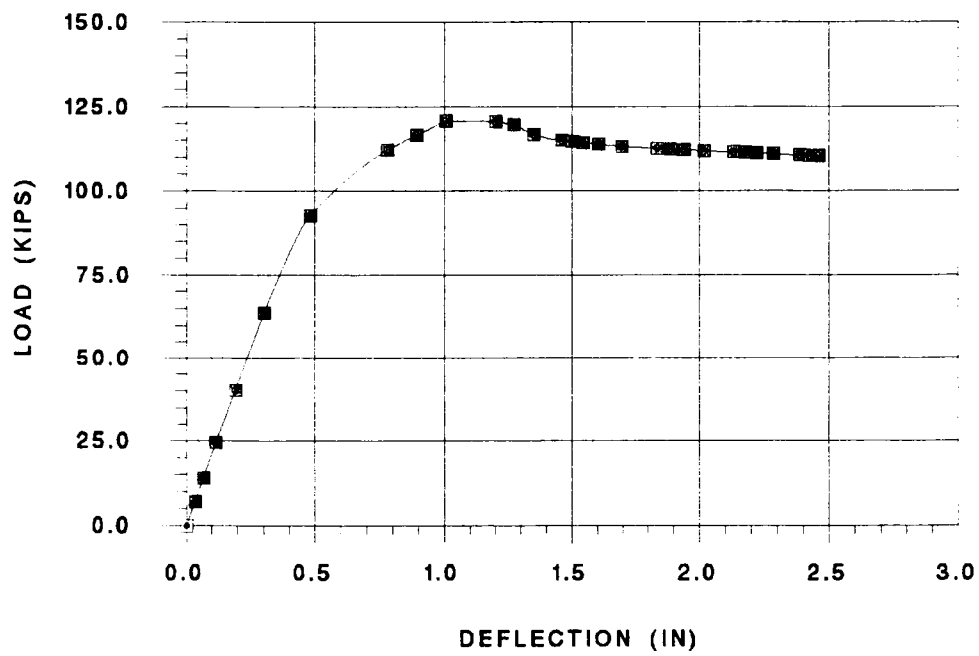
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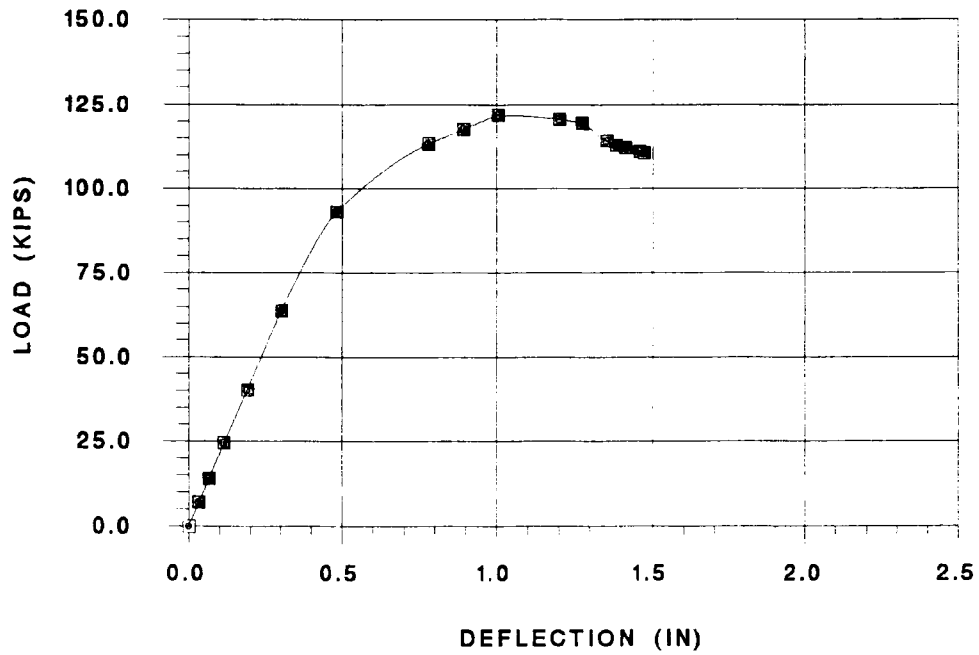
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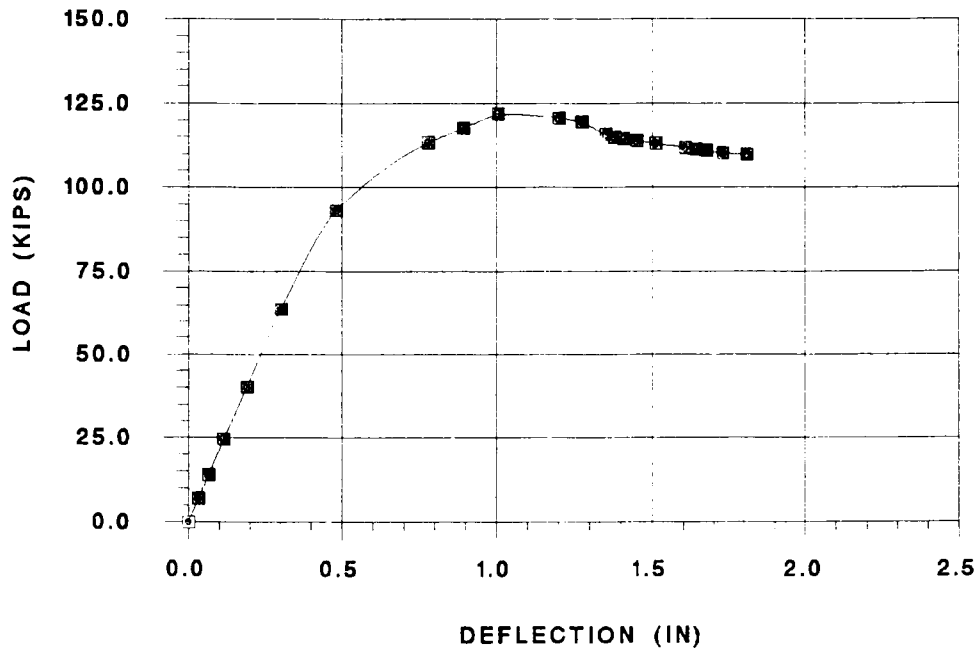
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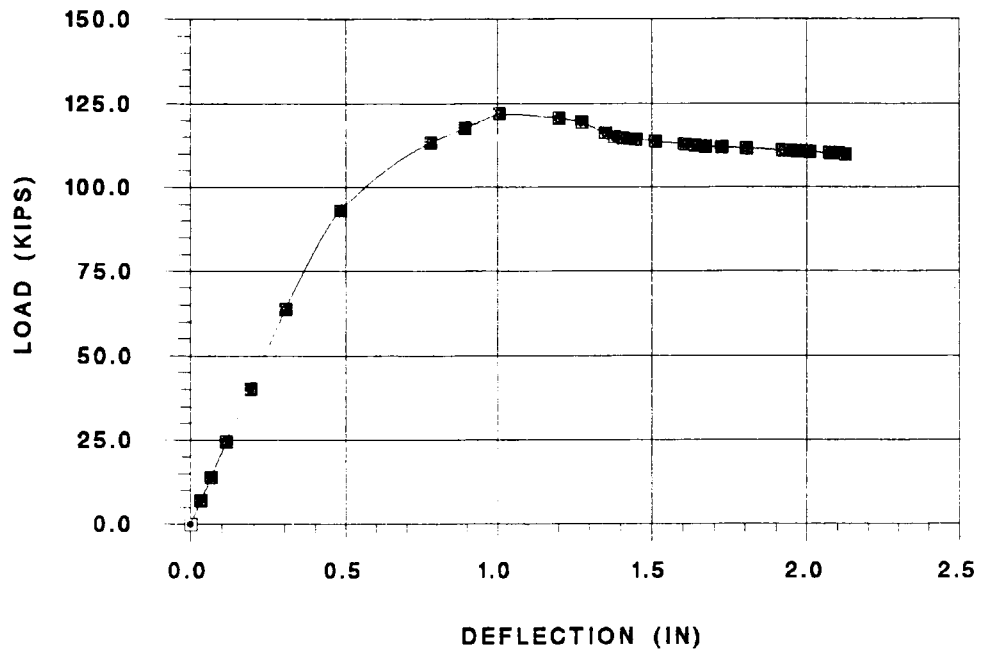
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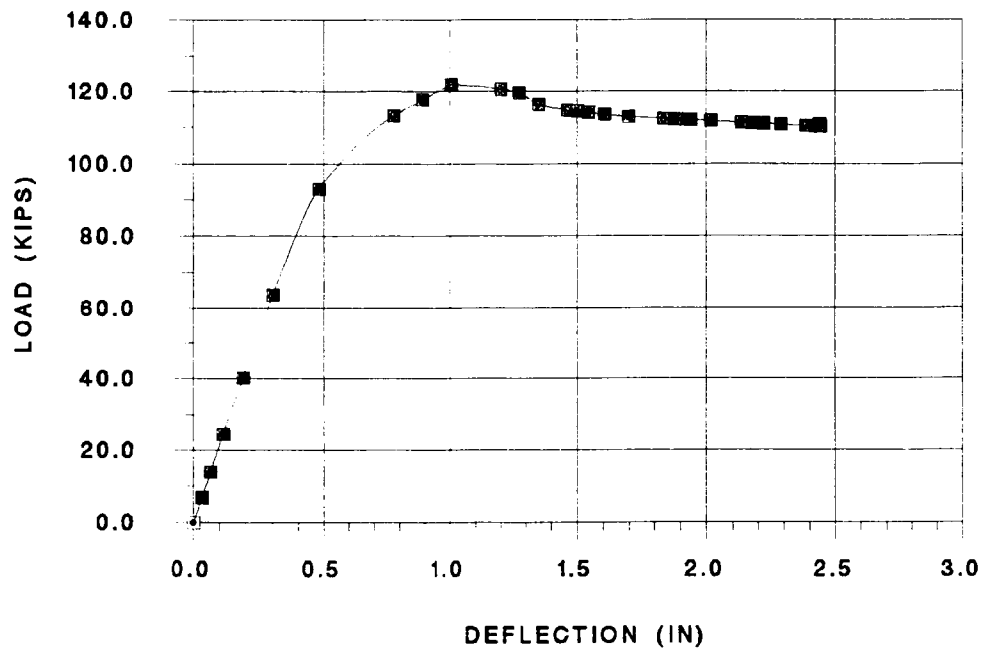
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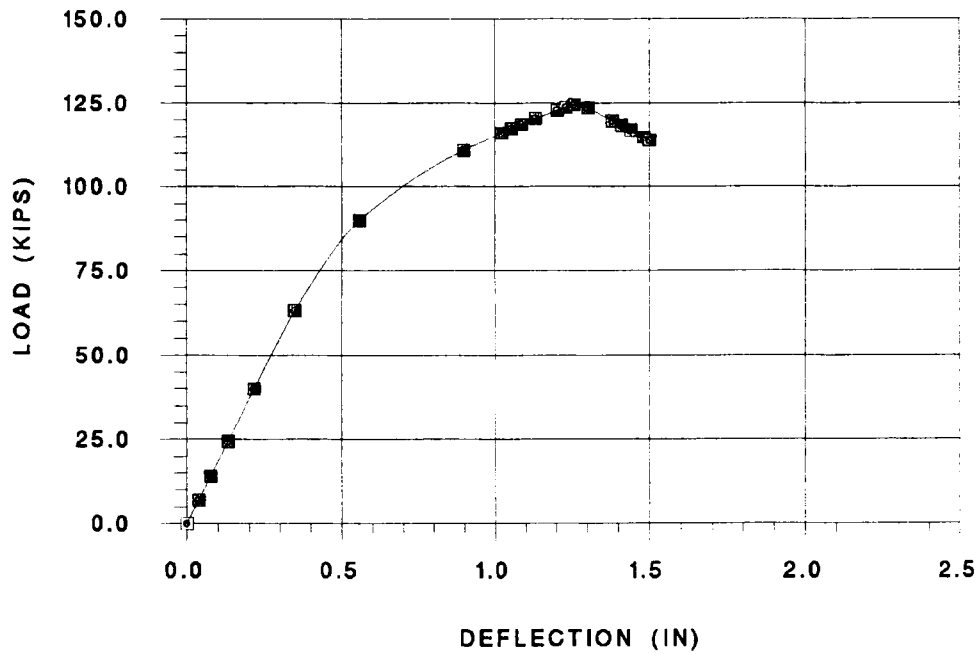
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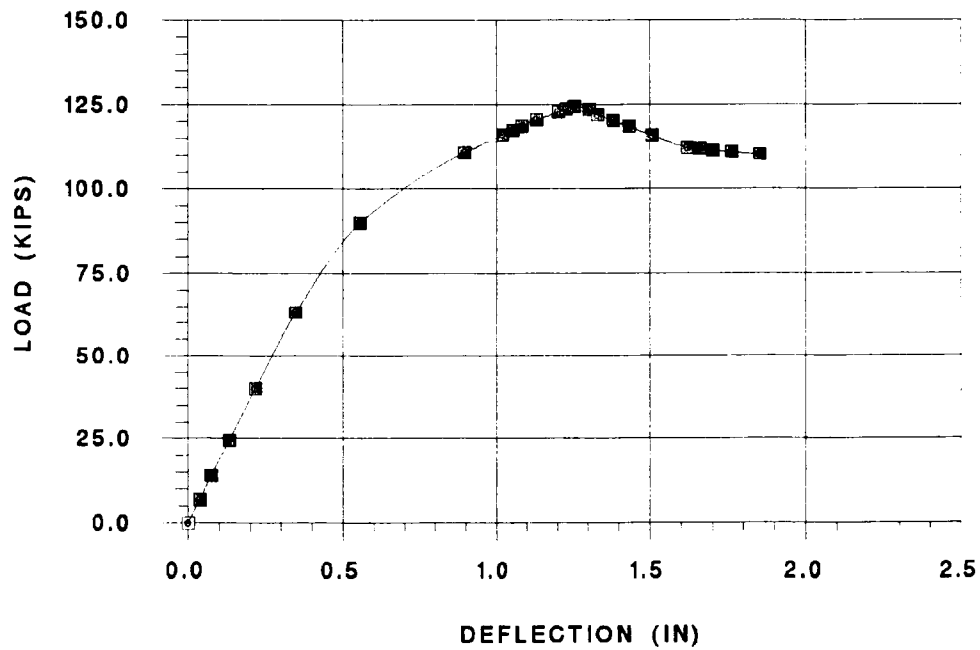
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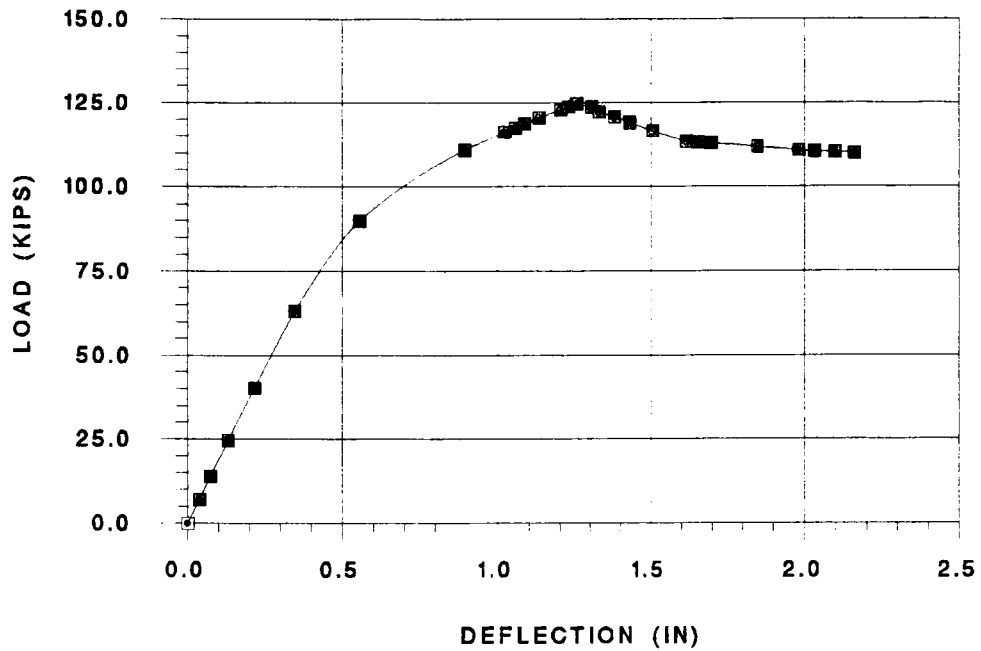
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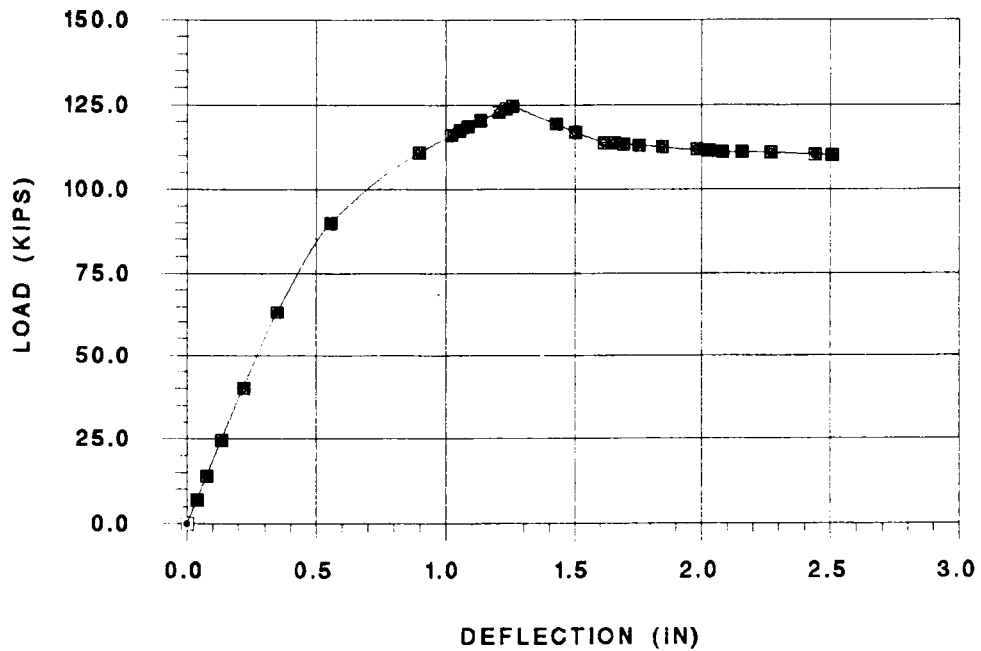
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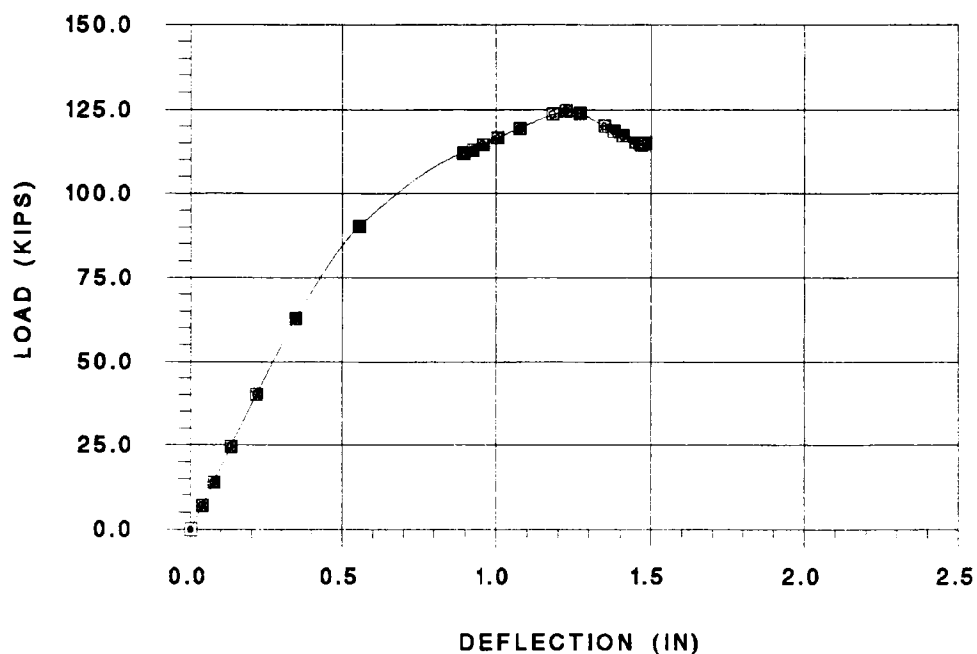
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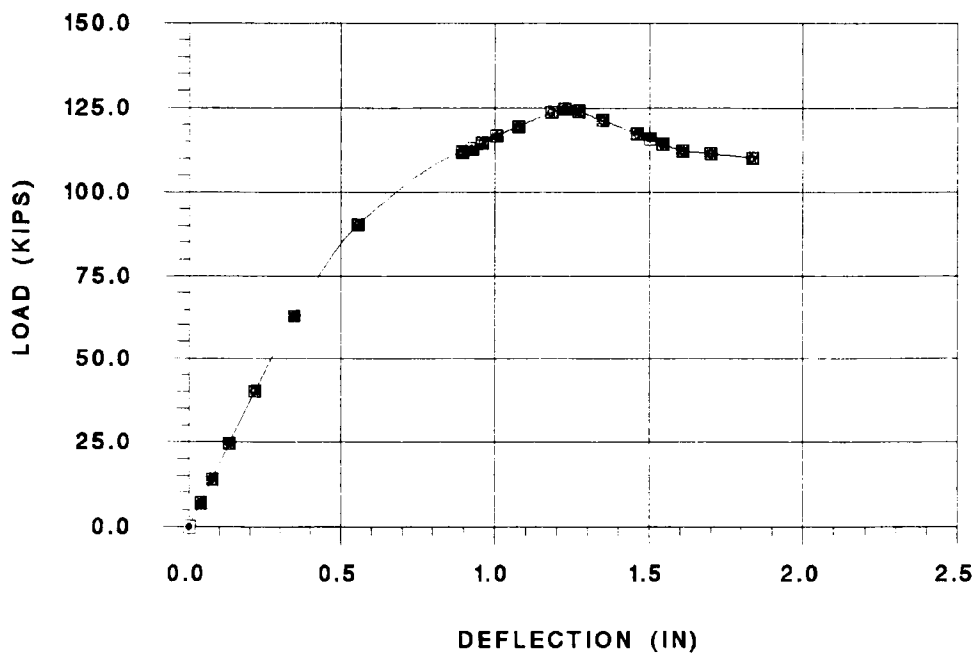
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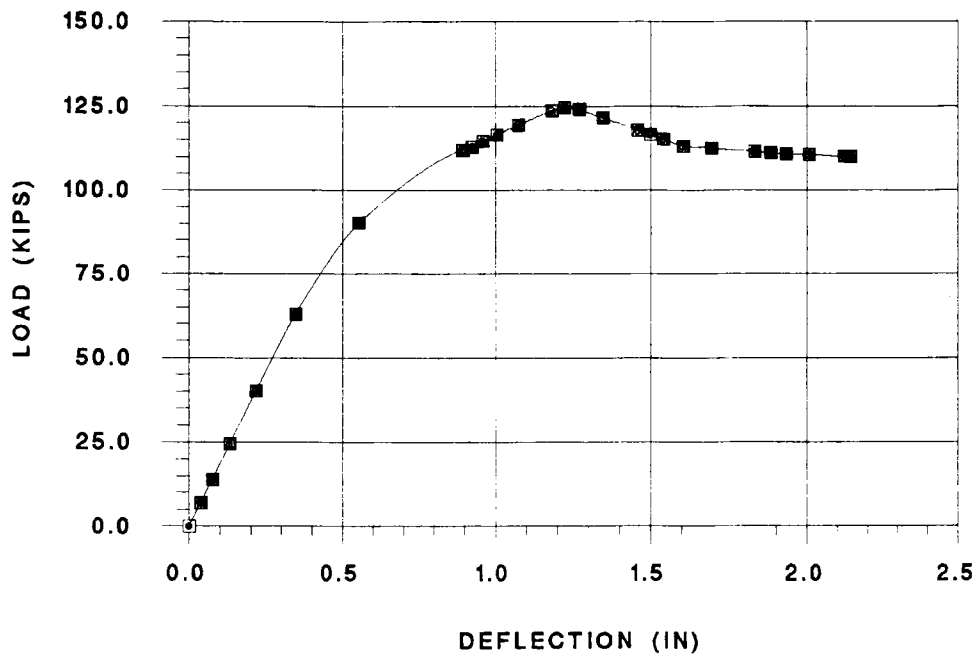
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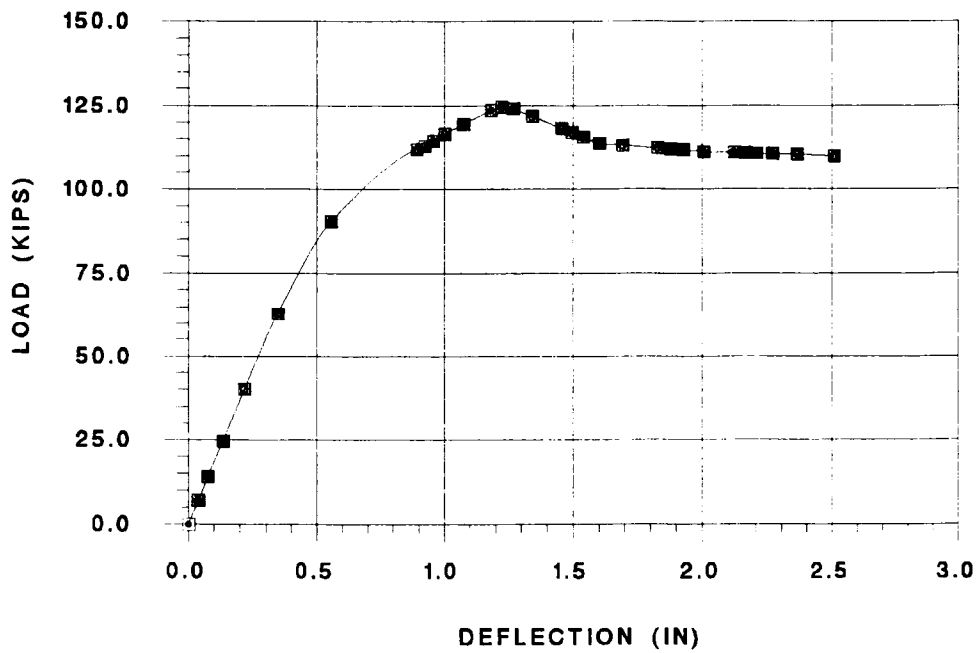
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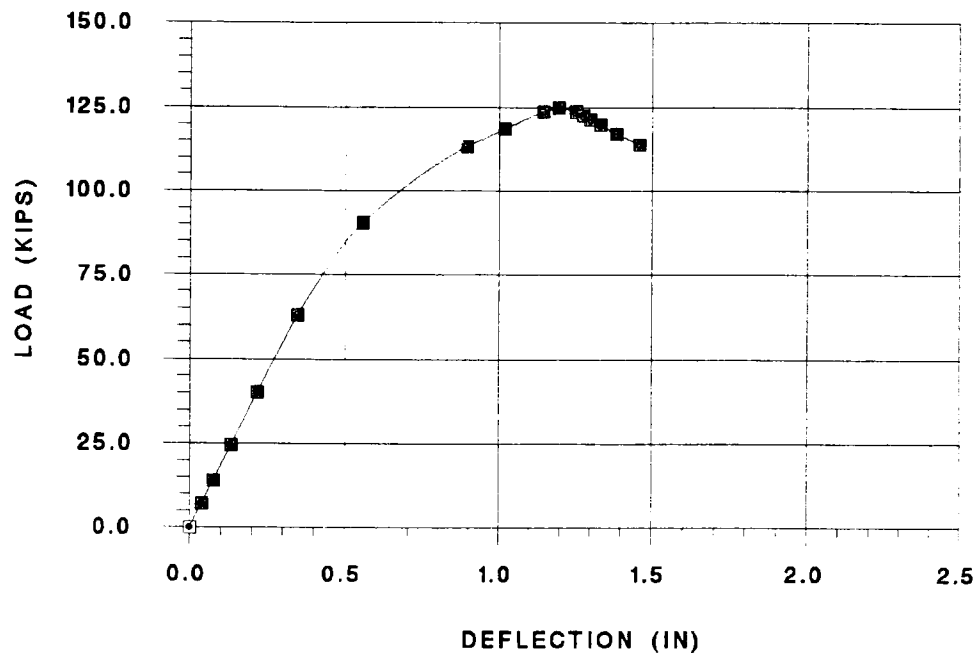
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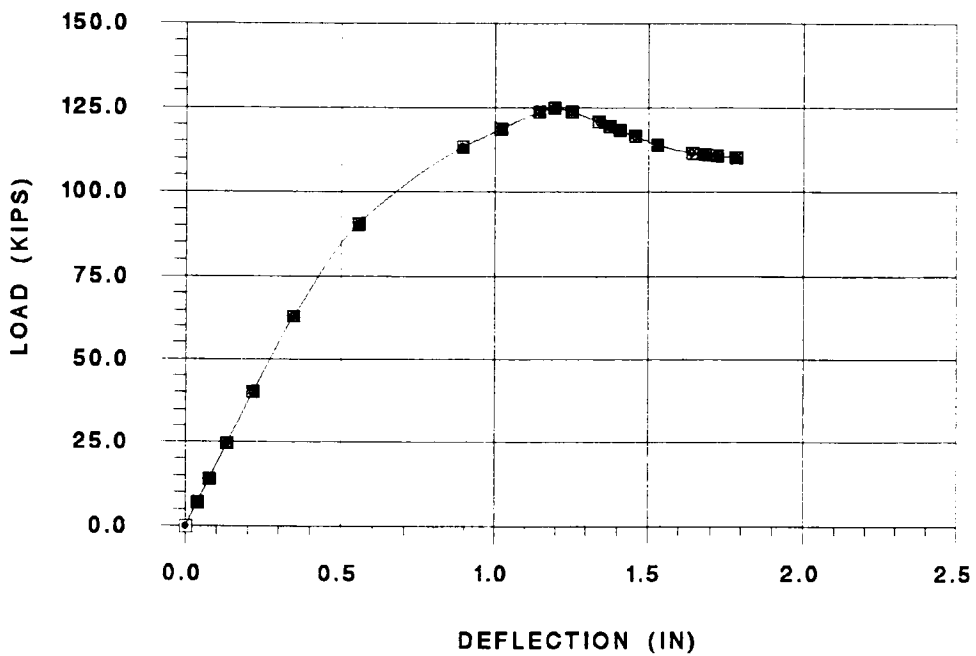
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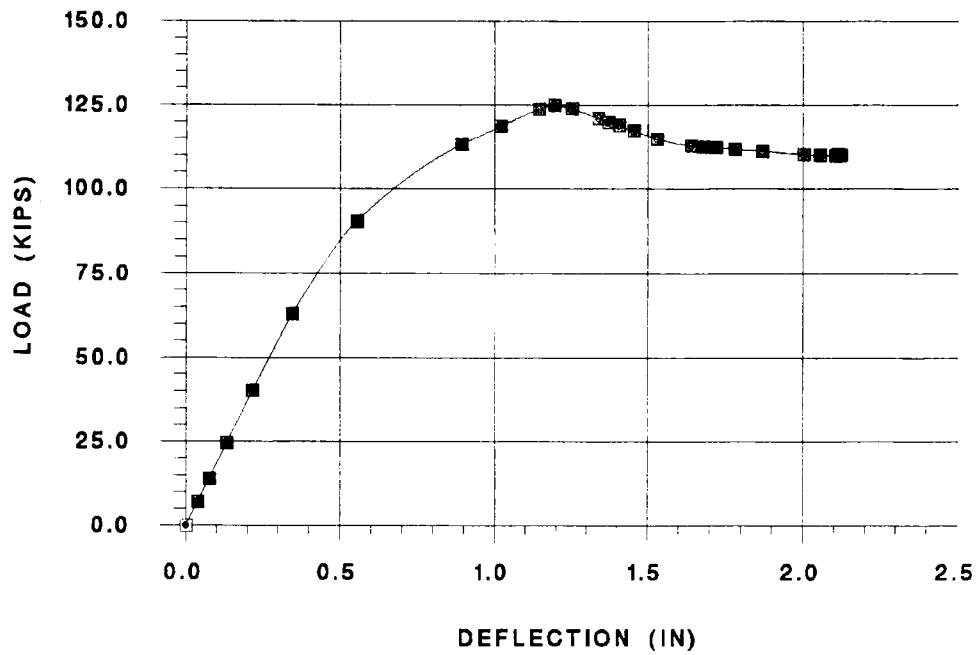
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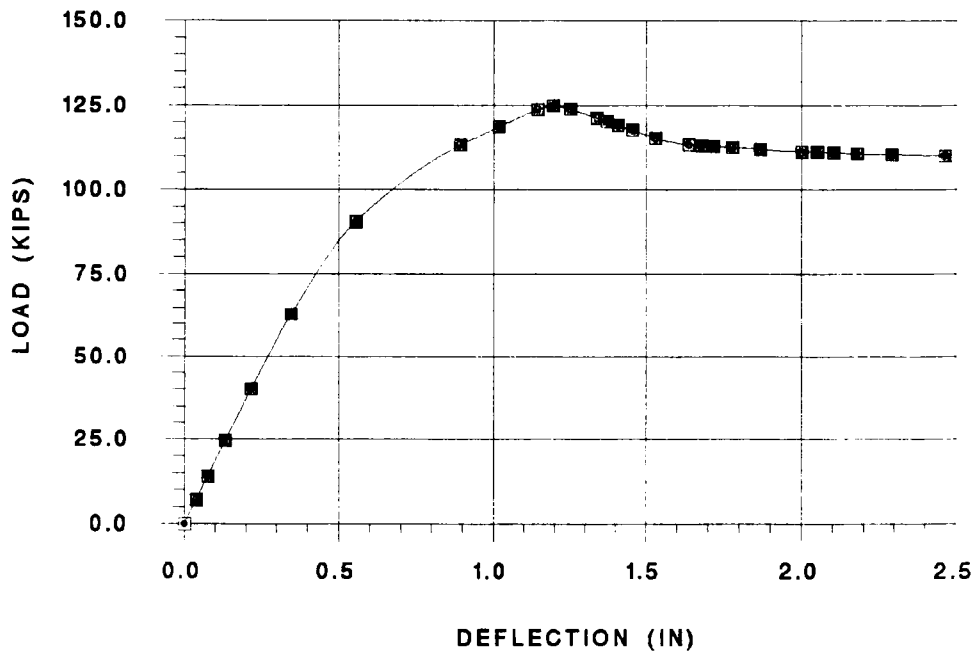
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Load vs. Deflection for Beam 5-SWAI-EAST-200-006



LOAD DEFLECTION DATA FOR BEAM 5-3-EXT-150

STRAIN = .003			STRAIN = .004			STRAIN = .005			STRAIN = .006		
LOAD (KIPS)	DEFLECTION (IN)		LOAD (KIPS)	DEFLECTION (IN)		LOAD (KIPS)	DEFLECTION (IN)		LOAD (KIPS)	DEFLECTION (IN)	
0.000	0.000		0.00	0.000		0.00	0.000		0.00	0.000	
6.500	0.031		6.50	0.031		0.00	0.000		0.00	0.000	
13.000	0.061		13.00	0.061		0.00	0.000		0.00	0.000	
22.750	0.107		22.75	0.107		6.50	0.031		6.50	0.031	
37.380	0.176		37.38	0.176		13.00	0.061		13.00	0.061	
59.310	0.279		59.31	0.279		22.75	0.107		22.75	0.107	
83.280	0.453		83.28	0.453		37.38	0.176		37.38	0.176	
97.280	0.733		97.28	0.733		59.31	0.279		59.31	0.279	
110.400	1.141		110.40	1.141		83.28	0.453		83.28	0.453	
114.700	1.294		114.70	1.294		97.28	0.733		97.28	0.733	
118.400	1.440		118.40	1.440		110.40	1.141		110.40	1.141	
119.800	1.495		119.80	1.495		114.70	1.294		114.70	1.294	
120.300	1.558		120.30	1.558		118.40	1.440		118.40	1.440	
119.200	1.643		119.00	1.659		120.30	1.558		120.30	1.558	
118.900	1.668		117.00	1.702		117.20	1.702		117.20	1.699	
116.100	1.702		116.10	1.744		116.40	1.742		116.40	1.743	
114.400	1.746		115.20	1.803		114.50	1.921		114.50	1.892	
112.300	1.810		113.90	1.892		114.20	1.954		114.20	1.956	
111.600	1.834		113.40	1.925		112.60	2.186		112.60	2.188	
111.700	1.839		112.90	1.959		112.40	2.228		112.40	2.230	
111.700	1.840		112.20	2.009		112.20	2.269		112.20	2.270	
111.800	1.841		111.50	2.083		111.80	2.330		111.80	2.331	
111.900	1.842		110.80	2.193		111.10	2.457		111.10	2.460	
112.000	1.845		110.80	2.193		111.00	2.492		111.05	2.495	
112.200	1.849		110.80	2.193					110.88	2.498	
112.300	1.850								110.80	2.612	
112.400	1.853								110.73	2.722	
112.500	1.853								110.65	2.762	
111.900	1.856								110.60	2.812	

LOAD DEFLECTION DATA FOR BEAM 5-3-EXT-191

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7.00	0.033		7.00	0.033		7.00	0.033		7.00	0.033	
14.00	0.066		14.00	0.066		14.00	0.066		14.00	0.066	
24.50	0.115		24.50	0.115		24.50	0.115		24.50	0.115	
40.25	0.190		40.25	0.190		40.25	0.190		40.25	0.190	
63.84	0.301		63.84	0.301		63.84	0.301		63.84	0.301	
92.30	0.482		92.30	0.482		92.30	0.482		92.30	0.482	
111.20	0.782		111.20	0.782		111.20	0.782		111.20	0.782	
115.60	0.896		115.60	0.896		115.60	0.896		115.60	0.896	
119.60	1.009		119.60	1.009		119.60	1.009		119.60	1.009	
120.90	1.194		120.90	1.194		120.90	1.194		120.90	1.194	
120.00	1.266		120.00	1.266		120.00	1.266		120.00	1.266	
117.10	1.342		117.80	1.341		118.00	1.340		118.10	1.340	
115.10	1.372		116.40	1.370		116.70	1.369		116.90	1.369	
113.60	1.402		115.00	1.399		115.50	1.398		115.70	1.398	
112.40	1.445		114.50	1.441		115.10	1.439		115.40	1.439	
110.70	1.510		113.70	1.503		114.50	1.501		114.90	1.501	
			112.40	1.597		113.60	1.594		114.10	1.593	
			112.00	1.632		113.30	1.629		113.90	1.628	
			111.50	1.667		112.90	1.664		113.60	1.663	
			110.80	1.720		112.40	1.717		113.20	1.715	
			110.20	1.799		112.10	1.795		112.90	1.793	
			110.00	1.828		111.50	1.912		112.50	1.909	
						111.20	1.956		112.30	1.953	
						111.00	2.000		112.20	1.997	
						110.70	2.065		111.90	2.063	
						110.30	2.164		111.60	2.161	
									111.10	2.309	
									110.90	2.364	
									110.70	2.419	
									110.50	2.502	

LOAD DEFLECTION DATA FOR BEAM 5-3-EXT-195

STRAIN = .003			STRAIN = .004			STRAIN = .005			STRAIN = .006		
LOAD (KIPS)	DEFLECTION (IN)		LOAD (KIPS)	DEFLECTION (IN)		LOAD (KIPS)	DEFLECTION (IN)		LOAD (KIPS)	DEFLECTION (IN)	
0.00	0.000		0.00	0.000		0.00	0.000		0.00	0.000	
7.00	0.033		7.00	0.033		7.00	0.033		7.00	0.033	
14.00	0.066		14.00	0.066		14.00	0.066		14.00	0.066	
24.50	0.115		24.50	0.115		24.50	0.115		24.50	0.115	
40.25	0.190		40.25	0.190		40.25	0.190		40.25	0.190	
63.83	0.301		63.83	0.301		63.83	0.301		63.83	0.301	
92.69	0.482		92.69	0.482		92.69	0.482		92.69	0.482	
112.20	0.781		112.20	0.781		112.20	0.781		112.20	0.781	
116.70	0.894		116.70	0.894		116.70	0.894		116.70	0.894	
120.80	1.007		120.80	1.007		120.80	1.007		120.80	1.007	
120.60	1.201		120.60	1.201		120.60	1.201		120.60	1.201	
119.70	1.273		119.70	1.273		119.70	1.273		119.70	1.273	
117.20	1.350		116.30	1.350		116.60	1.349		116.80	1.349	
114.90	1.379		114.90	1.379		115.40	1.378		115.00	1.460	
113.80	1.407		114.60	1.407		115.10	1.405		114.70	1.502	
112.50	1.448		114.00	1.448		114.70	1.447		114.30	1.543	
110.65	1.488		113.20	1.511		114.10	1.509		113.90	1.604	
			111.90	1.605		113.20	1.602		113.10	1.697	
			111.50	1.640		112.90	1.637		112.60	1.835	
			111.00	1.675		112.60	1.672		112.40	1.887	
			110.30	1.728		112.10	1.724		112.20	1.939	
			109.60	1.806		111.70	1.802		111.90	2.017	
						111.10	1.919		111.50	2.133	
						110.80	1.963		111.40	2.177	
						110.60	2.007		111.20	2.221	
						110.30	2.073		111.00	2.286	
						110.10	2.097		110.60	2.385	
						110.00	2.122		110.50	2.422	
						110.10	2.124		110.40	2.459	
						110.20	2.125				

LOAD DEFLECTION DATA FOR BEAM 5-3-EXT-200

STRAIN= .003			STRAIN = .004			STRAIN = .005			STRAIN = .006		
LOAD (KIPS)	DEFLECTION (IN)		LOAD (KIPS)	DEFLECTION (IN)		LOAD (KIPS)	DEFLECTION (IN)		LOAD (KIPS)	DEFLECTION (IN)	
0.00	0.000		0.00	0.000		0.00	0.000		0.00	0.000	
7.00	0.033		7.00	0.033		7.00	0.033		7.00	0.033	
14.00	0.066		14.00	0.066		14.00	0.066		14.00	0.066	
24.50	0.115		24.50	0.115		24.50	0.115		24.50	0.115	
40.25	0.190		40.25	0.190		40.25	0.190		40.25	0.190	
63.83	0.301		63.83	0.301		63.83	0.301		63.83	0.301	
93.15	0.481		93.15	0.481		93.15	0.481		93.15	0.481	
113.40	0.779		113.40	0.779		113.40	0.779		113.40	0.779	
117.80	0.891		117.80	0.891		117.80	0.891		117.80	0.891	
122.00	1.004		122.00	1.004		122.00	1.004		122.00	1.004	
120.70	1.201		120.70	1.201		120.70	1.201		120.70	1.201	
119.50	1.274		119.60	1.273		119.60	1.273		119.60	1.273	
114.30	1.353		115.90	1.351		116.30	1.350		116.40	1.350	
113.00	1.383		114.90	1.379		115.00	1.379		114.90	1.461	
112.20	1.412		114.50	1.407		114.80	1.406		114.50	1.502	
111.00	1.455		114.00	1.449		114.40	1.448		114.20	1.543	
110.60	1.471		113.10	1.511		113.80	1.510		113.70	1.605	
			111.90	1.605		112.90	1.603		113.00	1.698	
			111.40	1.640		112.60	1.638		112.50	1.836	
			111.00	1.675		112.20	1.672		112.30	1.888	
			110.30	1.728		112.10	1.724		112.10	1.940	
			109.80	1.806		111.70	1.802		111.80	2.017	
			109.90	1.806		111.10	1.919		111.40	2.134	
						110.90	1.963		111.20	2.178	
						110.60	2.007		111.10	2.221	
						110.30	2.073		110.50	2.385	
						110.20	2.098		110.40	2.422	
						110.00	2.122		110.50	2.434	
									110.60	2.436	
									110.30	2.441	

LOAD DEFLECTION DATA FOR BEAM 5-SWAI-EAST-190

STRAIN = .003			STRAIN = .004			STRAIN = .005			STRAIN = .006		
LOAD (KIPS)	DEFLECTION (IN)		LOAD (KIPS)	DEFLECTION (IN)		LOAD (KIPS)	DEFLECTION (IN)		LOAD (KIPS)	DEFLECTION (IN)	
0.00	0.000		0.00	0.000		0.00	0.000		0.00	0.000	
7.00	0.038		7.00	0.038		7.00	0.038		7.00	0.037	
14.00	0.075		14.00	0.075		14.00	0.075		14.00	0.075	
24.50	0.132		24.50	0.132		24.50	0.132		24.50	0.132	
40.25	0.217		40.25	0.217		40.25	0.217		40.25	0.217	
63.13	0.346		63.13	0.346		63.13	0.346		63.13	0.346	
90.00	0.555		90.00	0.555		90.00	0.555		90.00	0.555	
110.90	0.894		110.90	0.894		110.90	0.894		110.90	0.894	
116.20	1.021		116.20	1.021		116.20	1.021		116.20	1.021	
117.50	1.053		117.50	1.053		117.50	1.053		117.50	1.052	
118.70	1.084		118.70	1.084		118.70	1.084		118.70	1.084	
120.40	1.132		120.40	1.132		120.40	1.132		120.40	1.132	
122.90	1.202		122.90	1.202		122.90	1.202		122.90	1.202	
123.80	1.229		123.80	1.229		123.80	1.229		123.80	1.229	
124.70	1.255		124.70	1.255		124.70	1.255		124.70	1.255	
123.70	1.303		123.70	1.303		123.70	1.303		119.30	1.427	
119.80	1.381		122.10	1.331		122.300	1.329		117.00	1.502	
118.40	1.410		120.30	1.381		120.700	1.379		114.00	1.613	
116.90	1.439		118.50	1.432		119.100	1.429		113.70	1.653	
114.80	1.483		115.80	1.507		116.700	1.503		113.50	1.693	
113.90	1.500		112.40	1.619		113.500	1.614		113.10	1.753	
			112.00	1.659		113.200	1.654		112.60	1.843	
			111.60	1.700		112.900	1.695		111.80	1.978	
			111.00	1.761		111.800	1.846		111.50	2.029	
			110.40	1.851		110.800	1.981		111.30	2.080	
						110.500	2.032		111.10	2.155	
						110.300	2.095		110.90	2.269	
						110.000	2.159		110.40	2.439	
									110.30	2.503	

LOAD DEFLECTION DATA FOR BEAM 5-SWAI-EAST-195

STRAIN = .003			STRAIN = .004			STRAIN = .005			STRAIN = .006		
LOAD (KIPS)	DEFLECTION (IN)		LOAD (KIPS)	DEFLECTION (IN)		LOAD (KIPS)	DEFLECTION (IN)		LOAD (KIPS)	DEFLECTION (IN)	
0.00	0.000		0.00	0.000		0.00	0.000		0.00	0.000	
7.00	0.038		7.00	0.038		7.00	0.038		7.00	0.038	
14.00	0.075		14.00	0.075		14.00	0.075		14.00	0.075	
24.50	0.132		24.50	0.132		24.50	0.132		24.50	0.132	
40.25	0.217		40.25	0.217		40.25	0.217		40.25	0.217	
63.11	0.346		63.11	0.346		63.11	0.346		63.11	0.346	
90.33	0.554		90.33	0.554		90.33	0.554		90.33	0.554	
112.10	0.893		112.10	0.893		112.10	0.893		112.10	0.893	
113.00	0.925		113.00	0.925		113.00	0.925		113.00	0.925	
114.60	0.956		114.60	0.956		114.60	0.956		114.60	0.956	
116.70	1.003		116.70	1.003		116.70	1.003		116.70	1.003	
119.60	1.075		119.60	1.075		119.60	1.075		119.60	1.075	
123.60	1.182		123.60	1.182		123.60	1.182		123.60	1.182	
124.60	1.223		124.60	1.223		124.60	1.223		124.60	1.223	
123.90	1.270		124.00	1.270		124.10	1.270		124.10	1.270	
120.20	1.348		121.40	1.345		121.60	1.345		121.80	1.344	
118.70	1.377		117.30	1.458		118.00	1.456		118.30	1.456	
117.30	1.406		115.80	1.500		116.60	1.498		117.00	1.497	
115.20	1.450		114.30	1.542		115.20	1.540		115.70	1.539	
114.40	1.466		112.40	1.605		113.20	1.603		113.70	1.602	
114.50	1.470		111.50	1.697		112.50	1.694		113.20	1.692	
114.50	1.471		110.30	1.833		111.50	1.829		112.40	1.827	
114.60	1.471					111.20	1.880		112.10	1.878	
114.70	1.473					110.80	1.931		111.80	1.929	
114.80	1.474					110.60	2.007		111.30	2.005	
114.90	1.477					110.10	2.121		111.00	2.118	
115.20	1.481					110.10	2.132		110.90	2.161	
						110.00	2.142		110.80	2.203	
									110.40	2.363	
									110.00	2.507	

LOAD DEFLECTION DATA FOR BEAM 5-SWAI-EAST-200

STRAIN = .003			STRAIN = .004			STRAIN = .005			STRAIN = .006		
LOAD (KIPS)	DEFLECTION (IN)		LOAD (KIPS)	DEFLECTION (IN)		LOAD (KIPS)	DEFLECTION (IN)		LOAD (KIPS)	DEFLECTION (IN)	
0.000	0.00		0.00	0.000		0.00	0.000		0.00	0.000	
7.000	0.04		7.00	0.038		7.00	0.038		7.00	0.038	
14.000	0.08		14.00	0.075		14.00	0.075		14.00	0.075	
24.500	0.13		24.50	0.132		24.50	0.132		24.50	0.132	
40.250	0.22		40.25	0.217		40.25	0.217		40.25	0.217	
63.090	0.35		63.09	0.347		63.09	0.347		63.09	0.347	
90.540	0.55		90.54	0.553		90.54	0.553		90.54	0.553	
113.300	0.89		113.30	0.891		113.30	0.891		113.30	0.891	
118.700	1.02		118.70	1.017		118.70	1.017		118.70	1.017	
123.800	1.14		123.80	1.144		123.80	1.144		123.80	1.144	
124.900	1.19		124.90	1.194		124.90	1.194		124.90	1.194	
123.700	1.25		123.90	1.251		124.00	1.250		124.00	1.250	
122.600	1.27		120.80	1.339		121.10	1.338		121.30	1.338	
121.500	1.30		119.60	1.373		120.00	1.371		120.30	1.371	
119.800	1.33		118.40	1.406		119.00	1.404		119.20	1.404	
117.300	1.38		116.60	1.456		117.40	1.454		117.70	1.453	
114.000	1.46		114.00	1.530		114.90	1.528		115.40	1.527	
			111.70	1.641		112.90	1.638		113.50	1.636	
			111.30	1.681		112.60	1.678		113.20	1.676	
			110.90	1.722		112.30	1.718		113.00	1.716	
			110.30	1.782		111.90	1.778		112.60	1.776	
			110.30	1.783		111.30	1.869		112.10	1.866	
						110.30	2.004		111.30	2.002	
						110.10	2.055		111.20	2.052	
						109.90	2.106		111.00	2.103	
						110.10	2.110		110.80	2.178	
						110.30	2.114		110.50	2.292	
						110.10	2.122		110.10	2.462	

**LOAD DEFLECTION , STEEL STRESS-STRAIN, CONCRETE STRESS-STRAIN
DATA FOR BEAM 5-SWAI-EAST-190-003
(f'c = 15.0 KSI)**

LOAD (KIPS)	DEFLECTION (IN)	BOTTOM STEEL STRESS (KSI)	BOTTOM STEEL STRAIN (IN/IN)	TOP STEEL STRESS (KSI)	TOP/ CONCRETE STRAIN (IN/IN)	BOTTOM/CONC. STRESS (KSI)
0.00	0.000	190.0	0.006643	190.0	-0.000031	-1.5700
6.50	0.013	190.4	0.006657	189.5	-0.000051	-1.4500
13.00	0.027	190.8	0.006671	189.0	-0.000070	-1.2390
22.75	0.047	191.4	0.006692	188.3	-0.000100	-0.9911
37.38	0.076	192.3	0.006723	187.1	-0.000143	-0.6189
59.31	0.121	193.6	0.006770	185.5	-0.000209	-0.0606
92.22	0.189	195.6	0.006840	182.9	-0.000307	0.7768
134.20	0.296	202.5	0.007081	179.0	-0.000469	1.3110
151.70	0.488	239.3	0.008366	173.6	-0.000754	1.0210
146.00	0.795	275.0	0.011871	173.6	-0.000983	0.3335
124.10	1.100	275.0	0.016085	183.9	-0.000873	0.0000
122.10	1.397	275.0	0.019637	190.7	-0.000854	0.0000
120.60	1.843	275.0	0.024915	200.3	-0.000838	0.0000
119.20	2.511	275.0	0.032800	214.6	-0.000821	0.0000
118.70	3.513	275.0	0.044565	235.2	-0.000820	0.0000
118.90	5.016	275.0	0.062171	265.7	-0.000833	0.0000
119.00	7.270	275.0	0.088599	275.0	-0.000837	0.0000
119.00	10.653	275.0	0.128243	275.0	-0.000837	0.0000

**LOAD DEFLECTION , STEEL STRESS-STRAIN, CONCRETE STRESS-STRAIN
DATA FOR BEAM 5-SWAI-EAST-195-003
(f'c = 15.0 KSI)**

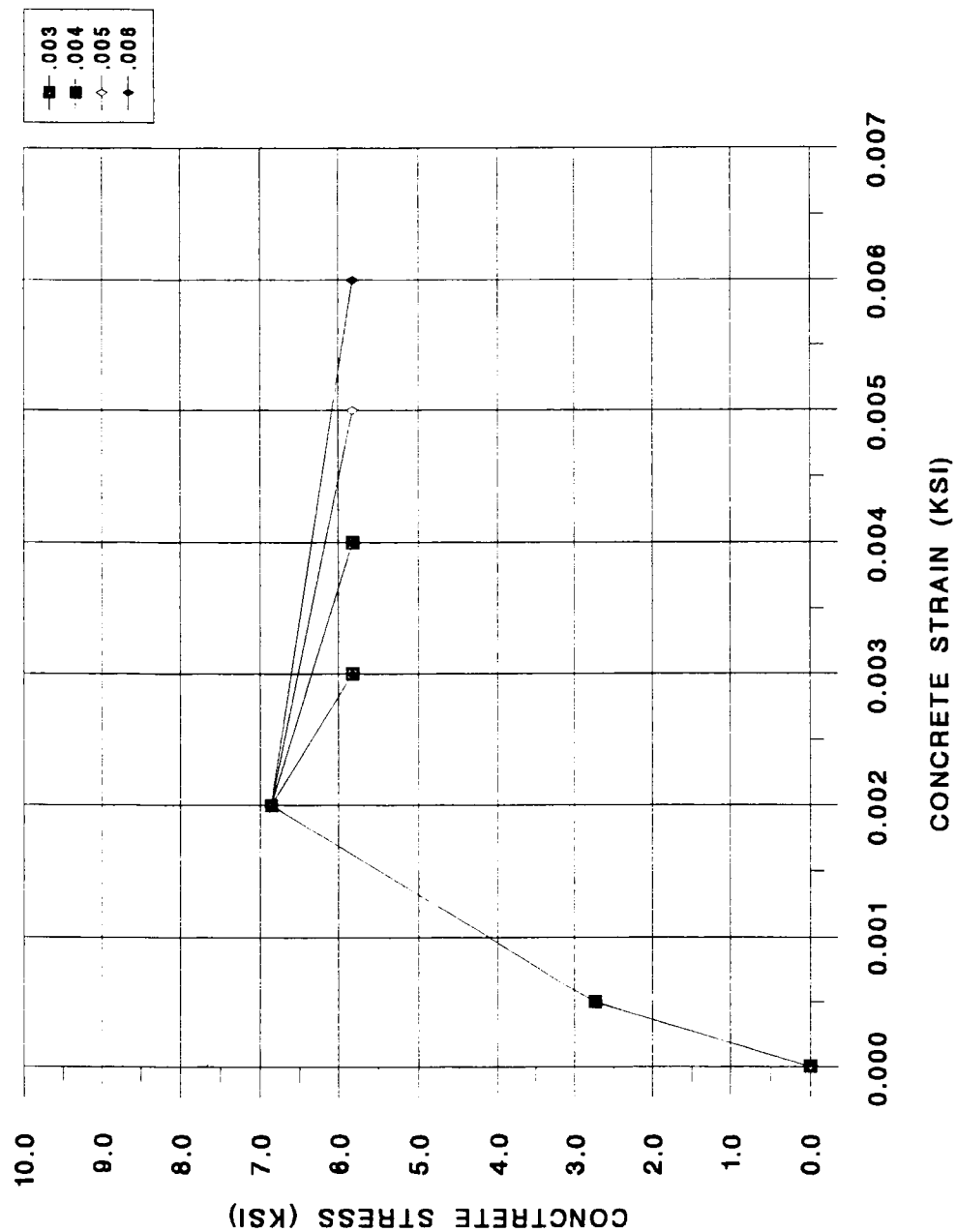
LOAD (KIPS)	DEFLECTION (IN)	BOTTOM STEEL STRESS (KSI)	BOTTOM STEEL STRAIN (IN/IN)	TOP STEEL STRESS (KSI)	TOP/CONCRETE STRAIN (IN/IN)	BOTTOM/CONC. STRESS (KSI)
0.00	0.000	195.0	0.006818	195.0	-0.000032	-1.615
6.50	0.013	195.0	0.006818	195.0	-0.000032	-1.615
13.00	0.027	195.4	0.006832	194.5	-0.000051	-1.450
22.75	0.047	195.8	0.006846	194.0	-0.000071	-1.284
37.38	0.076	196.4	0.006866	193.3	-0.000100	-1.036
59.31	0.121	197.3	0.006898	192.1	-0.000144	-0.6638
92.22	0.189	198.6	0.006944	190.5	-0.000209	-0.1055
135.00	0.296	200.6	0.007014	187.9	-0.000308	0.7319
153.50	0.488	207.3	0.007249	184.0	-0.000470	1.313
156.40	0.561	243.8	0.008524	178.4	-0.000761	1.026
160.10	0.633	258.8	0.009049	177.0	-0.000846	0.9232
159.00	0.751	270.5	0.009458	175.7	-0.000922	0.8429
139.60	0.870	275.0	0.010697	173.1	-0.001099	0.5997
128.00	0.984	275.0	0.012737	179.0	-0.001014	0.1991
123.90	1.153	275.0	0.014474	183.6	-0.000956	0.0000
122.30	1.404	275.0	0.016589	188.0	-0.000930	0.0000
120.80	1.780	275.0	0.019598	193.7	-0.000915	0.0000
119.60	2.344	275.0	0.030718	213.9	-0.000886	0.0000
118.70	3.189	275.0	0.040678	231.6	-0.000876	0.0000
118.80	4.457	275.0	0.055528	257.3	-0.000888	0.0000
119.00	6.360	275.0	0.077818	275.0	-0.000895	0.0000
119.00	9.213	275.0	0.111318	275.0	-0.000895	0.0000
119.00	13.495	275.0	0.161418	275.0	-0.000895	0.0000

**LOAD DEFLECTION , STEEL STRESS-STRAIN, CONCRETE STRESS-STRAIN
DATA FOR BEAM 5-SWAI-EAST-200-003
(f'c = 15.0 KSI)**

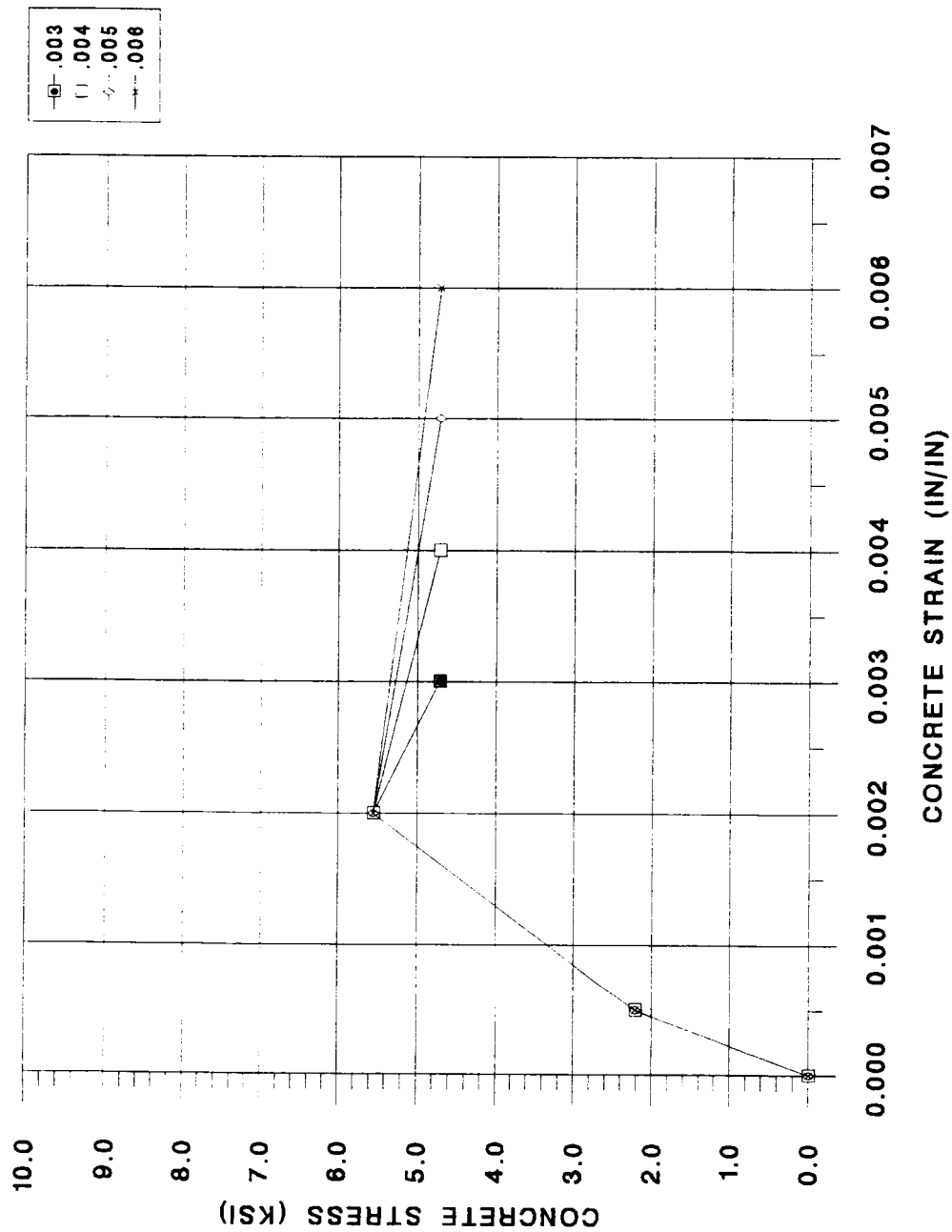
LOAD (KIPS)	DEFLECTION (IN)	BOTTOM STEEL STRESS (KSI)	BOTTOM STEEL STRAIN (IN/IN)	TOP STEEL STRESS (KSI)	TOP/ CONCRETE STRAIN (IN/IN)	BOTTOM/CONC. STRESS (KSI)
0.00	-0.000	200.0	0.006993	200.0	-0.000032	-1.6600
6.50	0.013	200.4	0.007007	199.5	-0.000052	-1.4940
13.00	0.027	200.8	0.007021	199.0	-0.000071	-1.3290
22.75	0.047	201.4	0.007041	198.3	-0.000100	-1.0810
37.38	0.076	202.3	0.007073	197.1	-0.000144	-0.7088
59.31	0.121	203.6	0.007119	195.5	-0.000210	-0.1505
92.22	0.189	205.6	0.007189	192.9	-0.000308	0.6869
135.80	0.295	212.2	0.007418	189.0	-0.000471	1.3150
155.20	0.487	248.0	0.008672	183.3	-0.000765	1.0340
145.30	0.798	275.0	0.012322	183.8	-0.000985	0.3163
124.20	1.103	275.0	0.016502	194.0	-0.000878	0.0000
122.20	1.400	275.0	0.020053	200.7	-0.000859	0.0000
120.70	1.846	275.0	0.025330	210.3	-0.000843	0.0000
119.30	2.514	275.0	0.033214	224.6	-0.000827	0.0000
118.80	3.516	275.0	0.044978	245.2	-0.000825	0.0000
119.00	5.019	275.0	0.062584	275.0	-0.000838	0.0000
119.00	7.273	275.0	0.089018	275.0	-0.000838	0.0000
119.00	10.657	275.0	0.128693	275.0	-0.000838	0.0000

Concrete Stress-Strain Curves

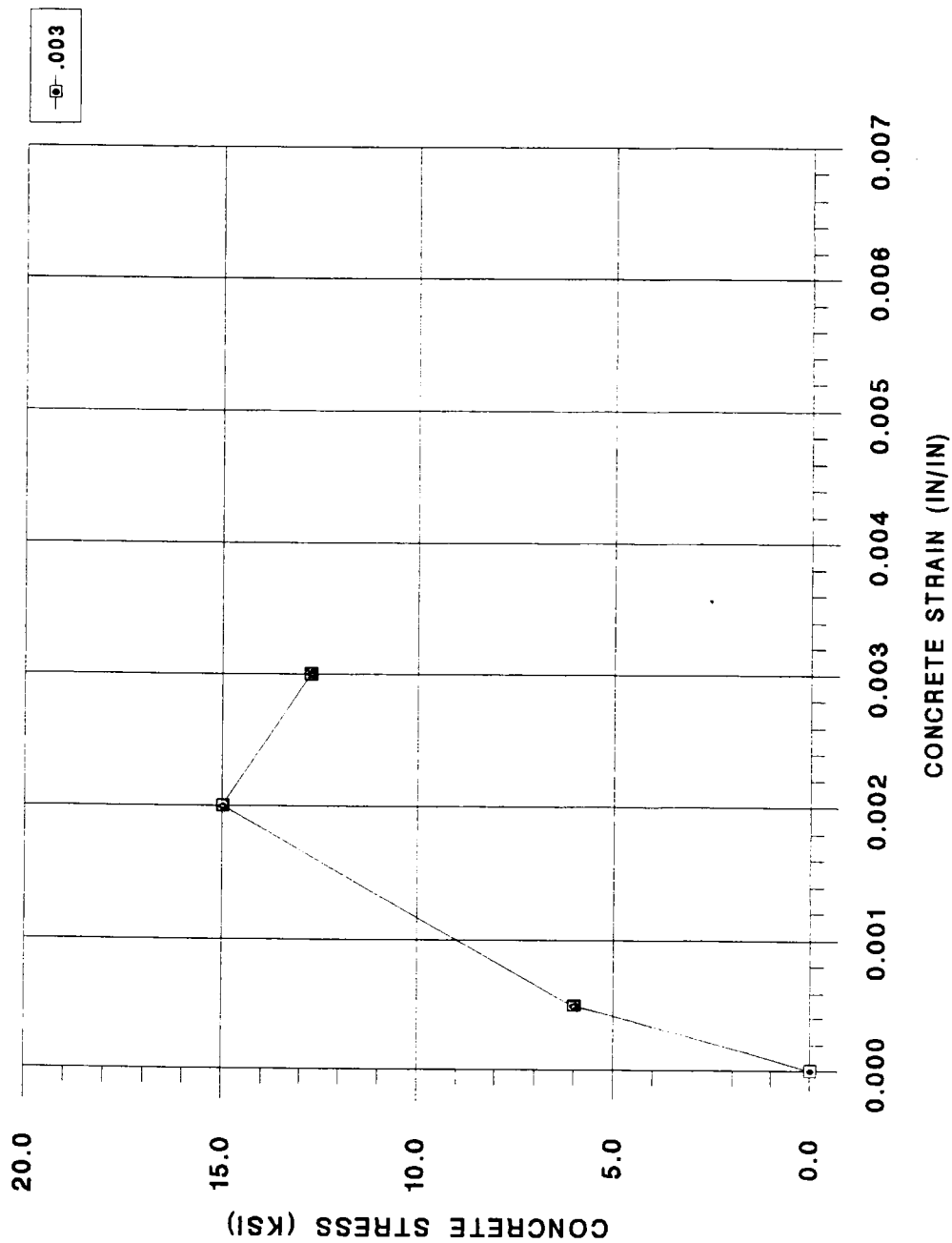
STRESS STRAIN CURVE FOR 6.86 KSI CONCRETE



STRESS STRAIN CURVE FOR 5.55 KSI CONCRETE



STRESS STRAIN CURVE FOR 15.0 KSI CONCRETE



Abaqus Sample Input

```
*HEADING
BEAM 5-3-EXT (FS=191, STRAIN=.006, STRESS=6.86 KSI)
*NODE,NSET=BOUND
1,0
29,28
56,55
87,85.1
113,112
141,140
169,168
197,196
225,224
253,252
281,280
309,308
337,336
365,364
373,372
*ELEMENT,TYPE=B21,ELSET=BM
1,1,29
2,29,56
3,56,87
4,87,113
5,113,141
6,141,169
7,169,197
8,197,225
9,225,253
10,253,281
11,281,309
12,309,337
13,337,365
14,365,373
*BEAM SECTION,SECTION=I-BEAM,MATERIAL=A1,ELSET=BM
0,28,16,12,7.12,5.33,6
*MATERIAL,NAME=A1
*ELASTIC
5.49E3,.18
*CONCRETE
2.74
6.86,.00075
5.83,.00494
*FAILURE RATIOS
1.16,.09
*TENSION STIFFENING
```

```

0.,6.678E-3
*REBAR,ELEMENT=BEAM,MATERIAL=A2,NAME=PRST
BM,1.224,0,3.75
*REBAR,ELEMENT=BEAM,MATERIAL=A2,NAME=TREI
BM,.306,0,24
*MATERIAL,NAME=A2
*ELASTIC
28.6E3
*PLASTIC
275
*INITIAL CONDITIONS,TYPE=STRESS(REBAR)
BM,PRST,191
BM,TREI,191
*BOUNDARY
1,PINNED
373,2
*STEP,INC=20,CYC=10,SUBMAX
*STATIC,PTOL=2.0
.05,1,,,,,87,2,-.315
*PRESTRESS HOLD
BM,PRST
BM,TREI
*DLOAD
BM,P2,-.024
*END STEP
*STEP
*STATIC,PTOL=10
.05,1,,,,,87,2,-2.0
*CLOAD
87,2,0
*NODE PRINT
U
CF
RF
*NODE FILE,NSET=BOUND
U,CF
*EL PRINT
S
E
SF
CRACK
*EL FILE,ELSET=BM,REBAR=PRST
S,E
*EL FILE,ELSET=BM,REBAR=TREI
S,E
*EL FILE,ELSET=BM
S
E

```

```

SF
CRACK
*END STEP
*STEP,INC=40,CYC=10,SUBMAX
*STATIC,PTOL=1.1,DIRECT,RIKS
.02,1,,,,,87,2,-10.0
*CLOAD
87,2,-130.
*NODE PRINT
U
CF
RF
*NODE FILE,NSET=BOUND
U,CF
*EL PRINT
S
E
SF
CRACK
*EL FILE,ELSET=BM,REBAR=PRST
S,E
*EL FILE,ELSET=BM,REBAR=TREI
S,E
*EL FILE,ELSET=BM
S
E
SF
CRACK
*END STEP

```

VITA

Jonathan Cyril Van Hook was born on May 8, 1966 in Knoxville, Tennessee. He attended grammar, junior high, and high school in the Rocky Hill, Bearden area in Knoxville. He graduated from Bearden High School in 1984.

The following September, he entered The University of Tennessee at Knoxville and in August, 1989 received the degree of Bachelor of Science in Civil Engineering.

He reentered The University of Tennessee at Knoxville the following term as a graduate assistant in Civil Engineering. Approximately two years later, He accepted a job with Technical Engineering Consultants, P.C. where he works today.

He graduated with a Master of Science degree in Civil Engineering (concentration in structural) in the summer of 1993.