FIELD OBSERVATION of FIVE LIGHTWEIGHT AGGREGATE PRETENSIONED PRESTRESSED CONCRETE BRIDGE BEAMS

Iowa Highway Research Board

Project HR 104

Iowa State Highway Commission

Ames, Iowa

ISHC

FOR BETTER HIGHWAYS

CORRECTION

HR-104, "Field Observations of Five Lightweight Aggregate Pretensioned Prestressed Concrete Bridge Beams."

Please replace Table #5, page 36 with the following.
(All Cambers are in inches)

BEAM NO.	INITIAL CAMBER PRED.	INITIAL CAMBER MEAS.	CAMBER PRIOR TO SLAB PLACEMENT PRED.	CAMBER PRIOR TO SLAB PLACEMENT MEAS.	CAMBER AFTER SLAB PLACEMENT PRED.	CAMBER AFTER SLAB PLACEMENT MEAS.	FINAL CAMBER (a) PRED.	FINAL CAMBER (a) MEAS.	
152	2.50	2.50	3.20	3.10	1.20	1.05	0.70	+ 0.25	
153	2.50	2.50	3.20	3.15	1.20	1.05	0.70	+ 0.40	
154	2.50	2.50	3.20	3.00	1.20	0.70	0.70	+ 0.20	
155	2.50	2.50	3.20	3.00	1.20	0.60	0.70	- 0.20	(b)
156	2.50	2.70	3.20	2.85	1.20	0.65	0.70	+ 0.10	

a) as of January 14, 1969

b) Figure 16, page 26 shows how the beam has developed a negative camber.

"Field Observation of Five Lightweight Aggregate Pretensioned Prestressed Concrete Bridge Beams

Final Report

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Ames, Iowa

Iowa Highway Research Board
Project No. HR-104

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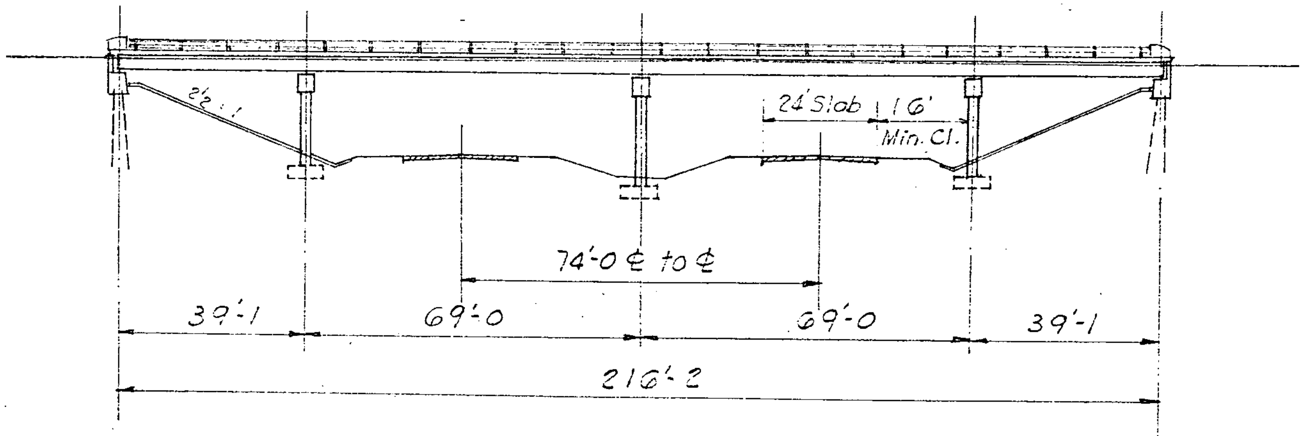
CHAPTER I

INTRODUCTION AND SCOPE

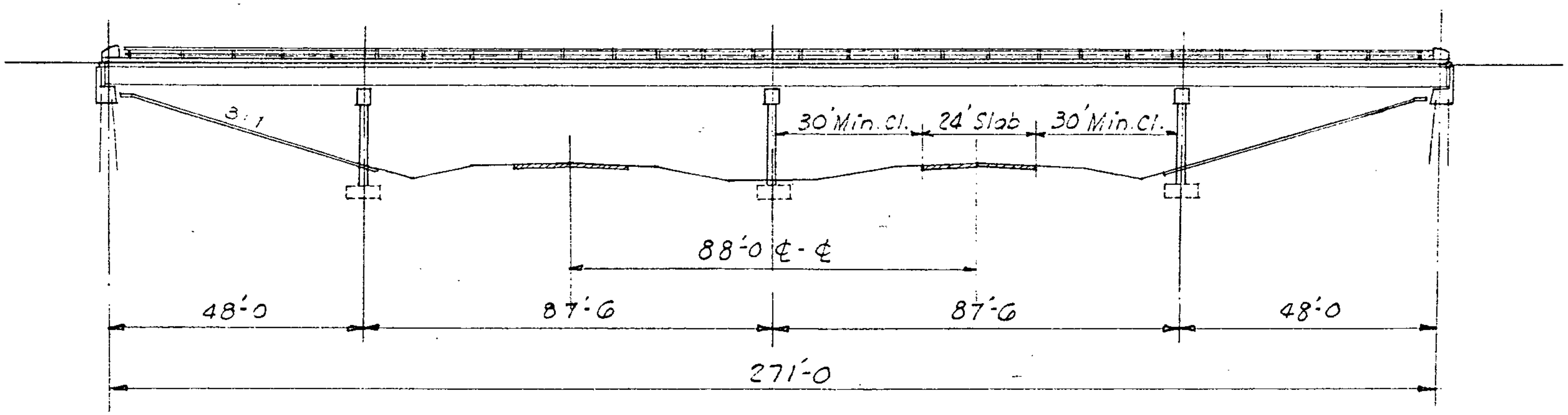
The use of lightweight aggregates in pretensioned prestressed concrete beams is becoming more advantageous as our design criteria dictate longer span concrete bridges. Bridge beams of greater lengths have been restricted from travel on many of our highways because the weight of the combined beams and transporting vehicle was excessive, making hauls of any distance prohibitive. This, along with the fact that new safety requirements necessitate the use of longer spans in grade separation structures over major highways, prompted the State of Iowa to investigate the use of lightweight aggregate bridge beams.

Until recently, it was possible to use 67' bridge beams in the two interior spans of a four span overhead crossing over interstate highways in Iowa. The new safety standards require that any obstruction such as columns or abutments be at least 30' beyond the outside edge of the pavement. This requirement means that beams for the two interior spans must be increased to at least 87' in length on a right angle crossing. If it should develop that a skewed crossing would be necessary, the length of the beams could conceivably be 90-95 feet in length. Figure I shows the relationship between typical new and old overhead crossing standards.

A series of three projects was started to investigate the possibility of using lightweight aggregate with natural sand fines in pretensioned prestressed concrete bridge beams. These projects were



Standard at Present



Proposed New Standard

Figure 1. Comparison of Old and Proposed New Standard Crossing

basically designed to investigate the feasibility of using lightweight aggregate bridge beams in the State of Iowa and to determine the properties of the material which are essential for design purposes. The three projects, which were started at approximately the same time are: "Creep and Shrinkage of Lightweight Aggregate Concretes," "Time Dependent Camber and Deflection of Non-Composite and Composite Lightweight Prestressed Beams," and "Field Observation of Five Lightweight Aggregate Pretensioned Prestressed Concrete Bridge Beams".

The first two are under the supervision of the Civil Engineering Department, at the University of Iowa, the third project is the subject of this report.

The objective of this project is the collection of field deflection measurements for five pretensioned prestressed lightweight aggregate concrete bridge beams fabricated by conventional plant processes; also the comparison of the actual cambers and deflections of the beams with that predicted from the design assumptions.

The test bridge is located on County Road "W" over Tipton Creek in Hardin County, Iowa. The bridge was designed by Mr. P. F. Barnard, Consulting Structural Engineer, Ames, Iowa and the beams were fabricated by Prestressed Concrete of Iowa, Inc., Iowa Falls, Iowa.

The Situation Plan and Superstructure Details of the test bridge are shown on pages A-1 and A-2 respectively in the Appendix.

CHAPTER II

EXPERIMENTAL PROCEDURES

2.1 Concrete Mix

One of the essential parts of any project involving concrete is the proper proportioning and mixing of the necessary constituents.

Three possible sources of lightweight aggregate were suggested for use on this project and they were tested by the Material Testing Laboratory at the Iowa Highway Commission. Aggregate A was eliminated on the basis of a very low durability of its beam samples. Aggregates B and C, using air dry aggregate, had durability factors of 100 and 97 respectively. It was noted, that even though aggregates B and C had durabilities which were acceptable, the beam made with aggregate B crumbled around the edges at one end. Based on these results Aggregate C, known by the brand name Idealite, was selected.

Table 1 shows the Mix Design Objectives, Ingredients and Procedures for this project.

2.2 Instrumentation

A permanent set of reference points was established on each beam at a distance of 22" from each end and at the midspan. The distance of 22" was used so that the reference points on the end would not be covered by the abutment diaphragms when the bridge is complete. These reference points consisted of 3½" x2" brass plates cast to the bottom flange of the beam. A ½" diameter hole

TABLE I CONCRETE MIX QUANTITIES FOR LIGHTWEIGHT CONCRETE BRIDGE BEAMS

MIX DESIGN OBJECTIVES	
Concrete Quantity	1½ cu. yds.
Concrete Strength @ 28 days	5000 psi
Unit Weight, Maximum Air-Dry	(117) pcf
Air Entrainment	(5± 1) %
MIX INGREDIENTS	
Cement (Type 1)	1058 lbs.
Natural Sand	2093 lbs.
Idealite Aggregates (60% of 3/4" to 5/16" and 40% of 5/16" to #8)	1230 lbs.
Water	52.5 gal.
Darex @ 7/8 oz. per sack of cement	9.75 oz.
Pozzolith	31.5 oz.

MIXING PROCEDURES

1. Proportion sand and Idealite.
2. Add 26 gallons of water.
3. Mix for approximately two minutes.
4. Proportion the cement.
5. Add six gallons of water.
6. Add Darex AEA in 3 gallons of water.
7. Add Pozzolith with remaining water while adjusting to 2½" slump.

was tapped in the plate; the hole was used to receive a ½" SAE - fine thread bolt on which the level rod is seated when the camber readings are taken. Figure 2 shows the position of the plates on the beam. A level rod (reading to 0.005') and a precise level was used on all measurements. The camber and deflection are determined on the basis of relative displacement between the reference points.

2.3 Beams

Five pretensioned prestressed concrete beams were cast in 2 groups; the first group of 3 was cast on April 15, 1968 and steam cured for 40 hours; the second group of 2 was cast on April 19th, 1968 and cured 67 hours in steam. The beam detail and data sheet is shown on page A-3 in the Appendix.

Group I, consisting of beams numbered 152, 153, & 154 was to have been released* on April 16th, however the cylinder strength did not reach 4500 fc' and the beams were not released until the 17th of April. Table 2 shows the strength and age of the cylinders at the time of testing.

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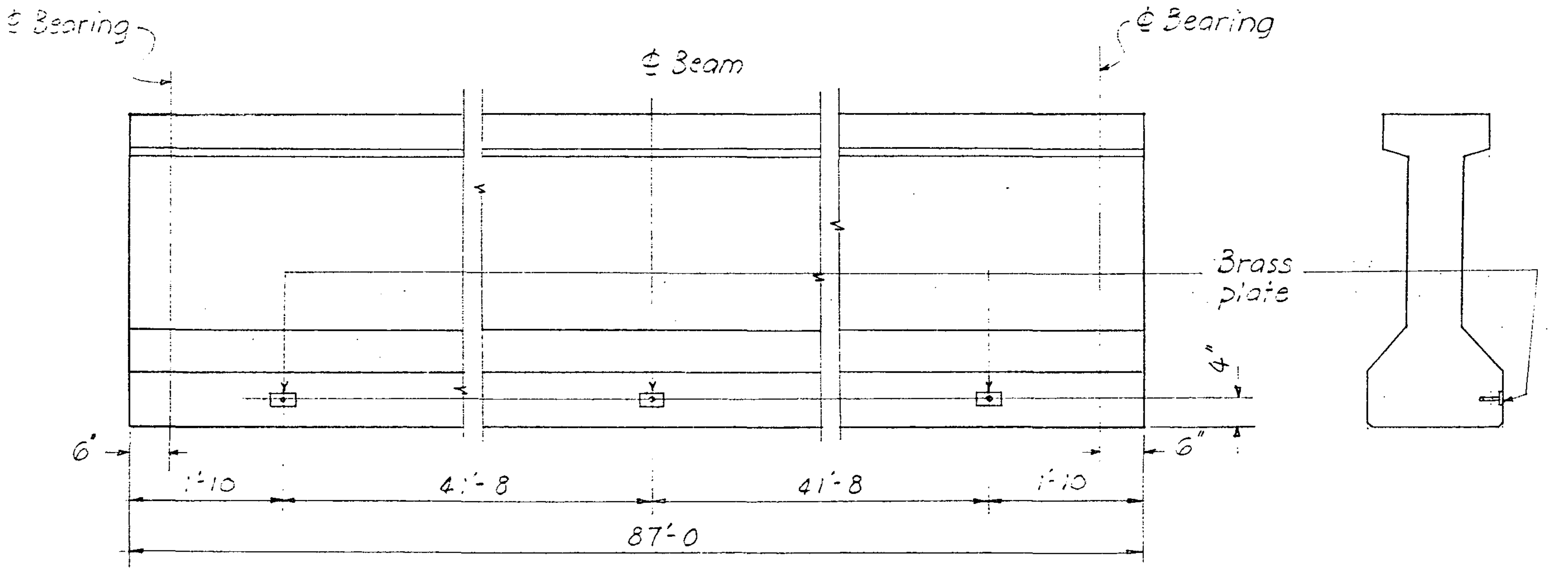
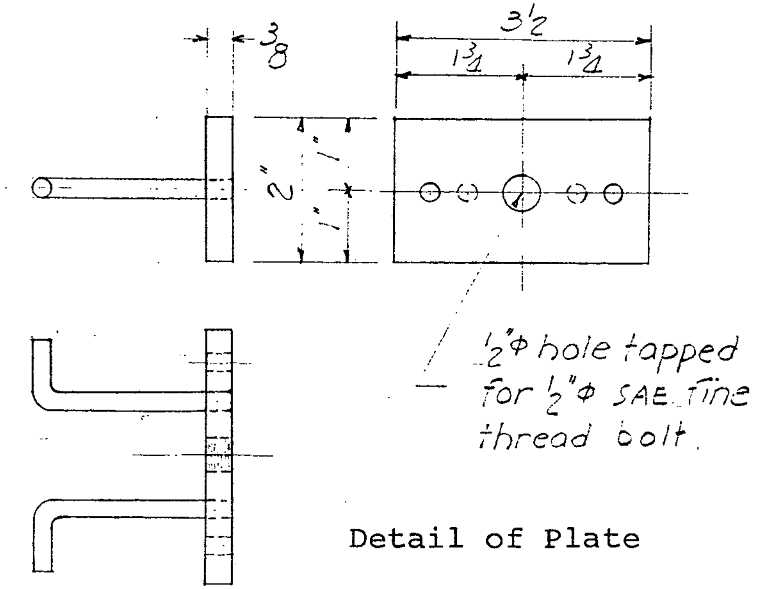
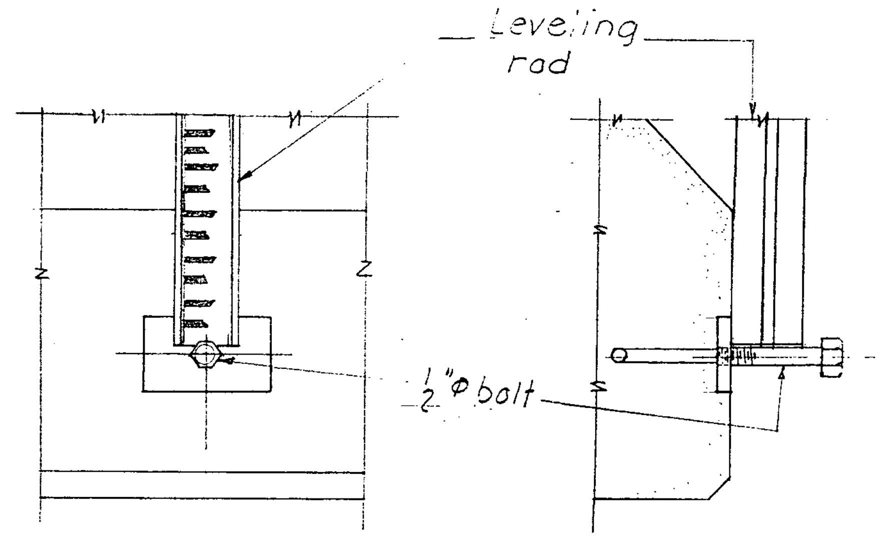
*released - is defined as the time at which the pretensioned cables are cut and the stress is transferred from the prestressing steel to the concrete.





Detail of Plate



Rod position on Reference Bolt

Figure 2, Detail and Location of Brass Plates

Table 2, STRENGTH AND AGE OF CYLINDERS FOR GROUP I BEAMS

Cylinder No.[a]	Beam No.	Age (Hours)	Strength (psi)
152A	152	24	4310
152B	152	48.5	5160
153A	153	40.5	4460
153B	153	48.5	4480
154A	154	40.5	4420
154B	154	48.5	4950

[a] Note: All cylinders except 152A were tested at time of release.

Group II, consisting of beams numbered 155 and 156, was cast on the 19th, and released on the 22nd of April. At the time of release, the test cylinders exhibited the strengths and ages as shown in Table 3.

Table 3, STRENGTH AND AGE OF CYLINDERS FOR GROUP II BEAMS

Cylinder No.	Beam	Age (Hours)	Strength (psi)
155A	155	67	5130
155B	155	-	-
156A	156	67	4350
156B	156	67	4360

When the beams were released, readings were taken at short intervals of time to show the development of camber with respect to time. The graphs in figures 3-7 show the camber development immediately after release. Figure 8 shows how the camber developed in beam number 155 over very small increments of time immediately after release. During the first minute and 45 seconds the beam camber held at a constant value of about 0.10 inch. It would appear that the bond and friction between the beam and steel pallet restrained

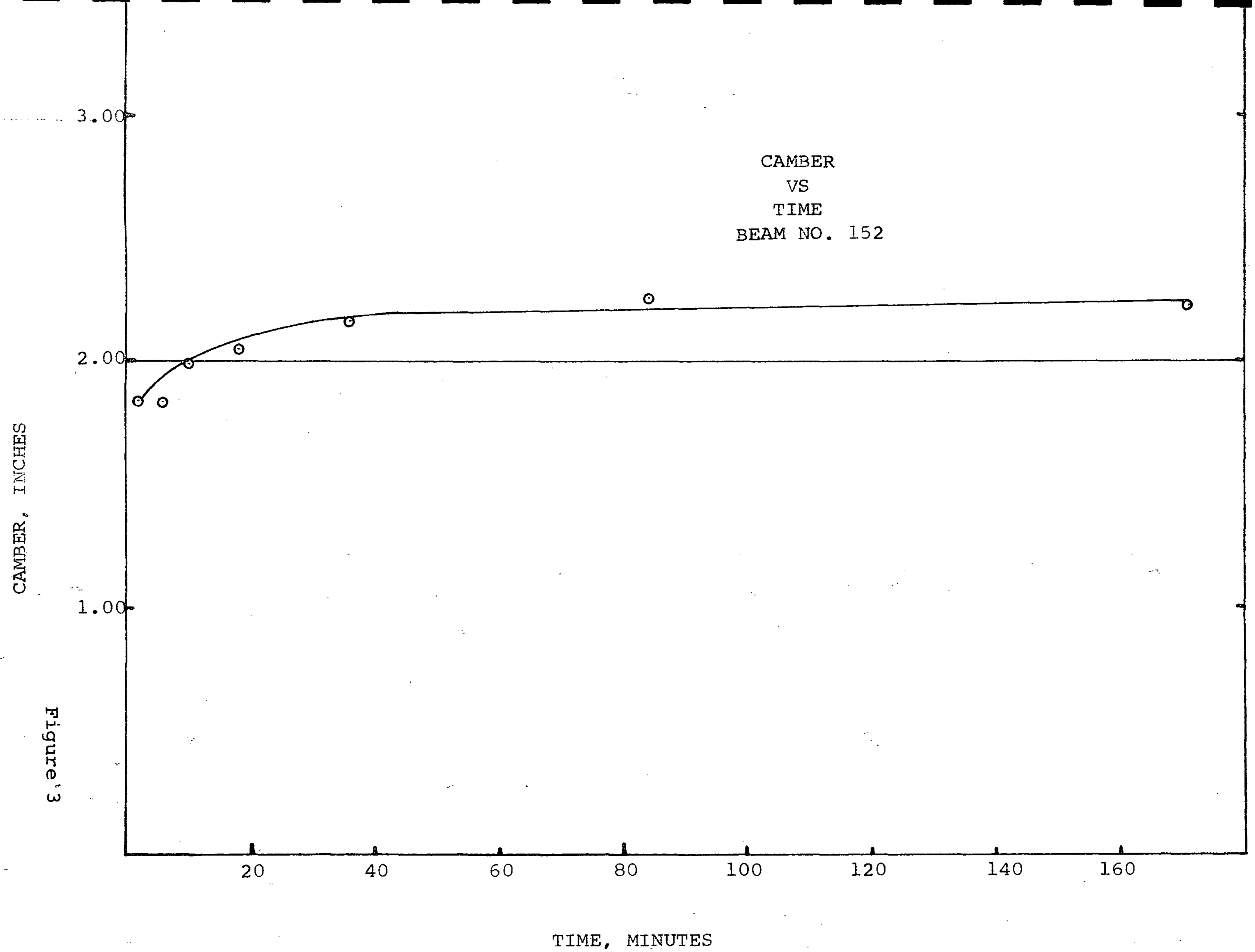

CAMBER
VS
TIME
BEAM NO. 152
3.00
2.00
1.00
CAMBER, INCHES
20
40
60
80
100
120
140
160
TIME, MINUTES

Figure 3

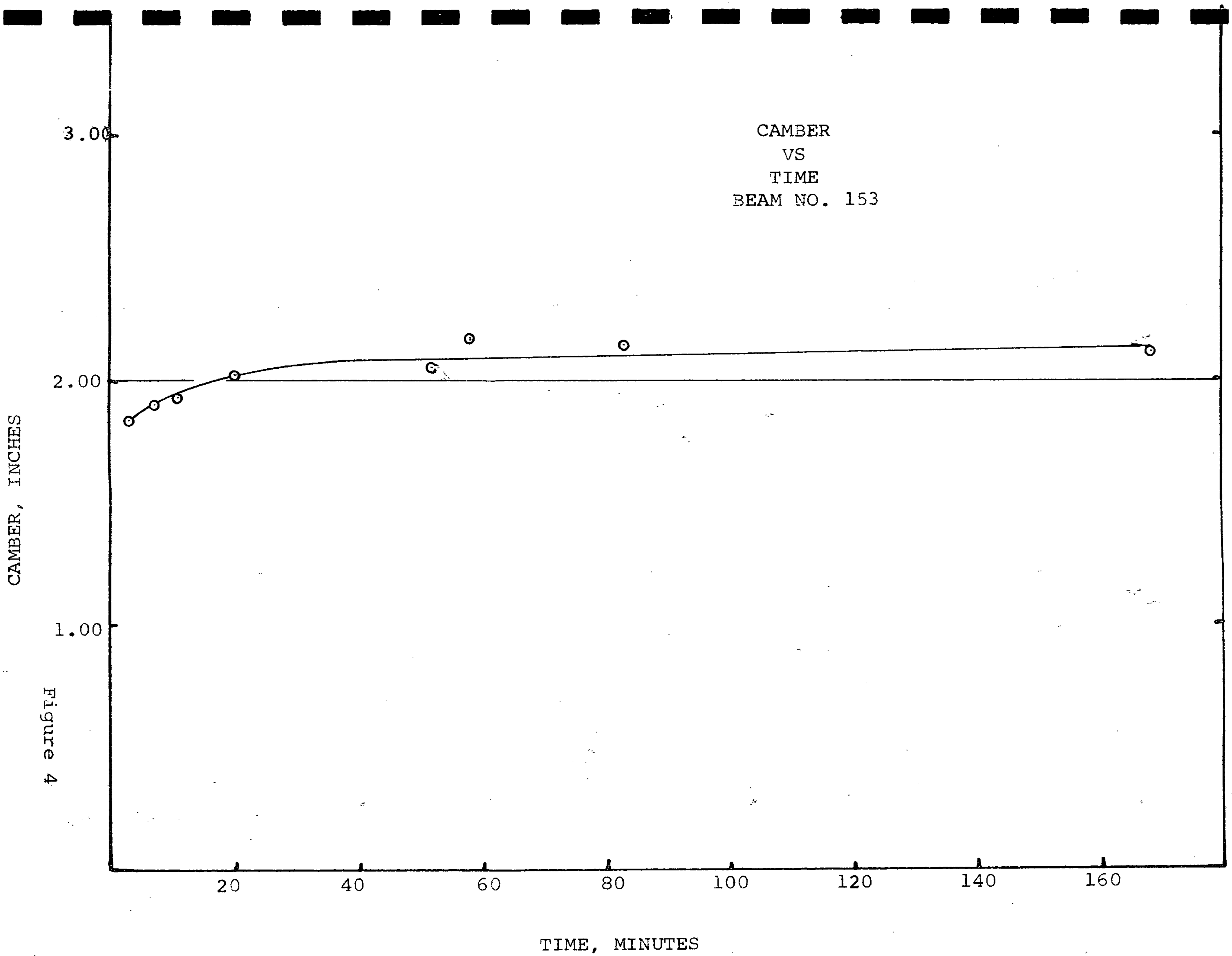

CAMBER
VS
TIME
BEAM NO. 153
CAMBER, INCHES
3.00
2.00
1.00
20
40
60
80
100
120
140
160
TIME, MINUTES

Figure 4

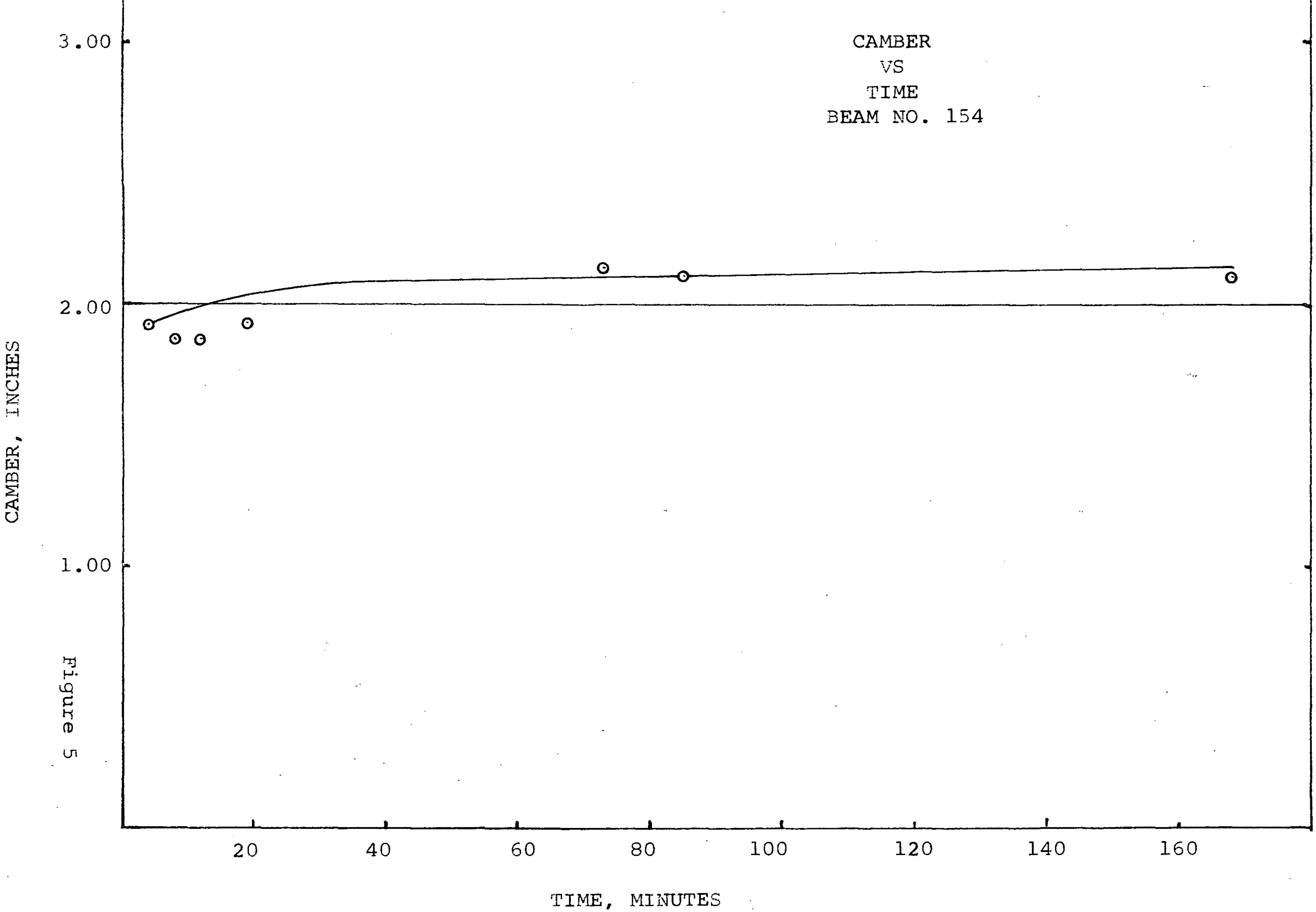

CAMBER
VS
TIME
BEAM NO. 154
3.00
2.00
1.00
CAMBER, INCHES
20
40
60
80
100
120
140
160
TIME, MINUTES

Figure 5

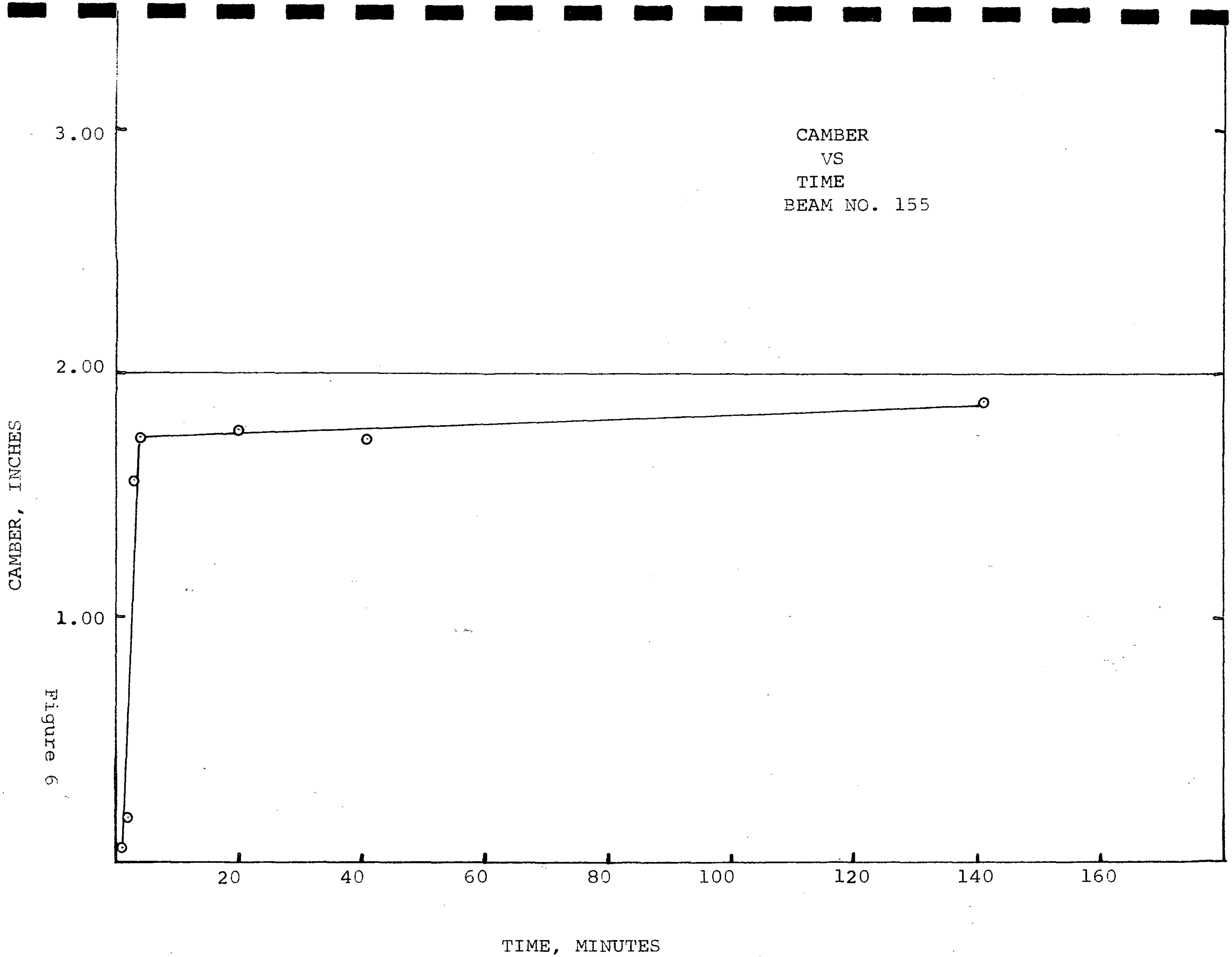

CAMBER
VS
TIME
BEAM NO. 155
3.00
2.00
1.00
CAMBER, INCHES
20
40
60
80
100
120
140
160
TIME, MINUTES

Figure 6

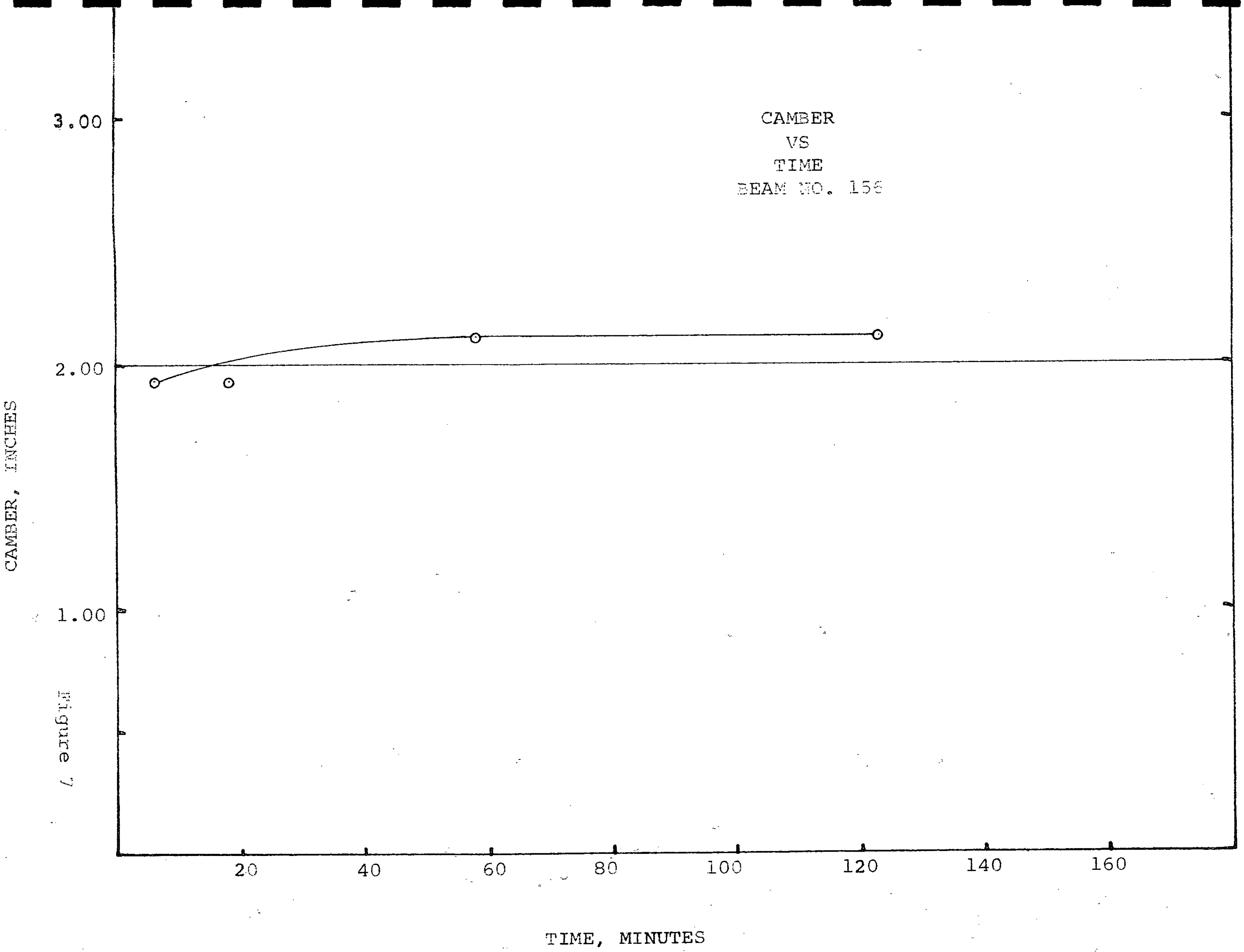

CAMBER
VS
TIME
BEAM NO. 156
3.00
2.00
1.00
CAMBER, INCHES
20
40
60
80
100
120
140
160
TIME, MINUTES

Figure 7

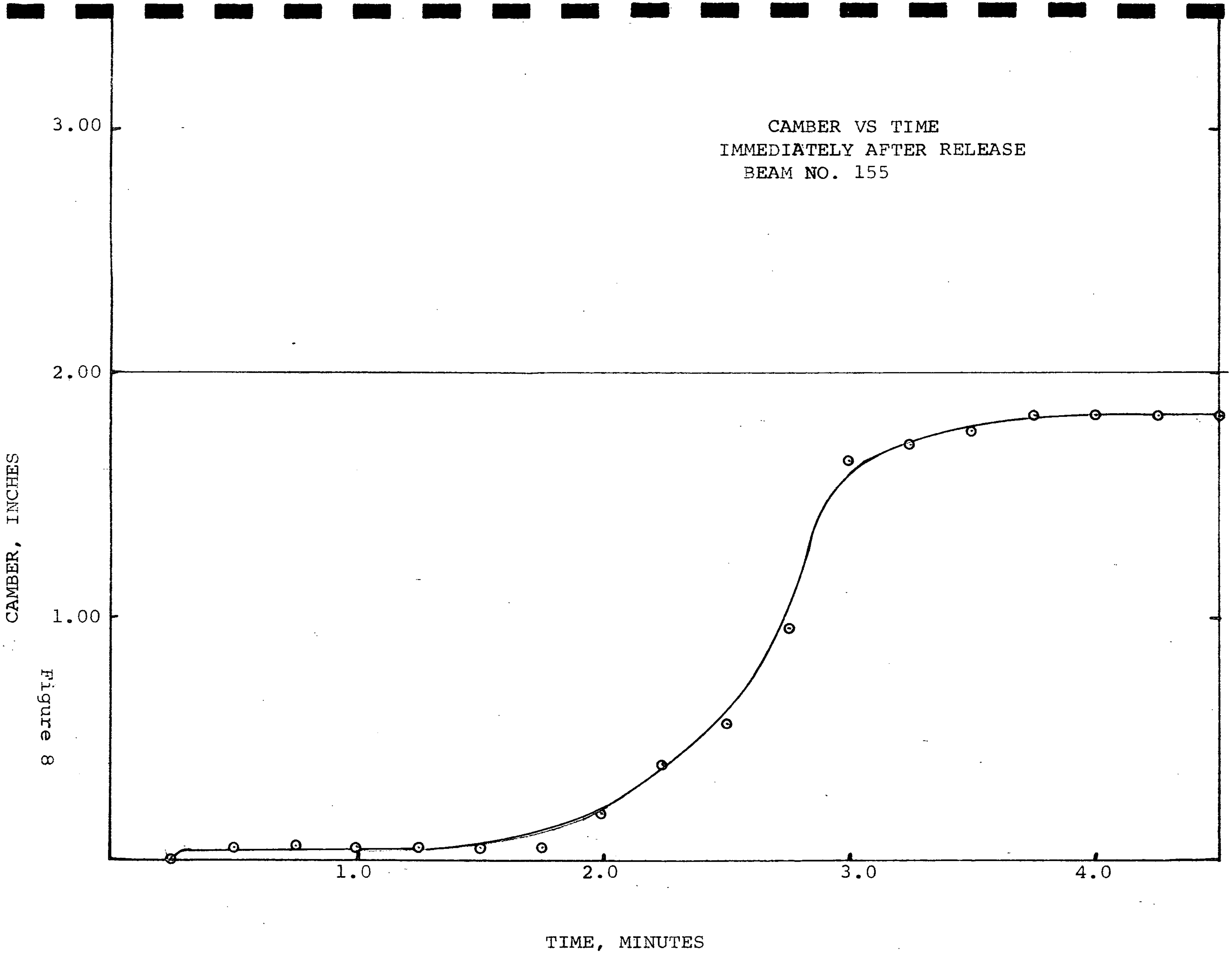

CAMBER VS TIME
IMMEDIATELY AFTER RELEASE
BEAM NO. 155
3.00
2.00
1.00
CAMBER, INCHES
1.0
2.0
3.0
4.0
TIME, MINUTES

Figure 8

the beam from deflecting upwards during this interval of time. Once the beam overcame the bond and friction deflection upwards was rapid. The deflection proceeded to increase until it leveled off at about 1.80" only 2 minutes after the bottom flange was separated from the pallet.

During the 2 months immediately after casting, the beams were stored outside. The temperature varied during this period from a low of 30^{o}F on the 24th of April to a high of 89^{o}F on the 7th of June. Camber measurements were taken during the period of 22 April to 7 June at an average of once every 8 days.

On June 10th the girders were moved from their plant storage location to the bridge site. A final set of readings was taken after the beams were seated on the abutments and before any superimposed load was applied. Figure 9 shows the beam layout on the test bridge.

On June 21st the deck was placed on the bridge. As the deck concrete placement progressed, deflection measurements were taken at 30 minute intervals. The placement of the deck took approximately five hours without any major delays. Readings were taken at 30 minute intervals over this entire period.

2.4 Field Deflection Measurements

The observation of the deflections of the center of the beams is one of the prime objectives of this report.

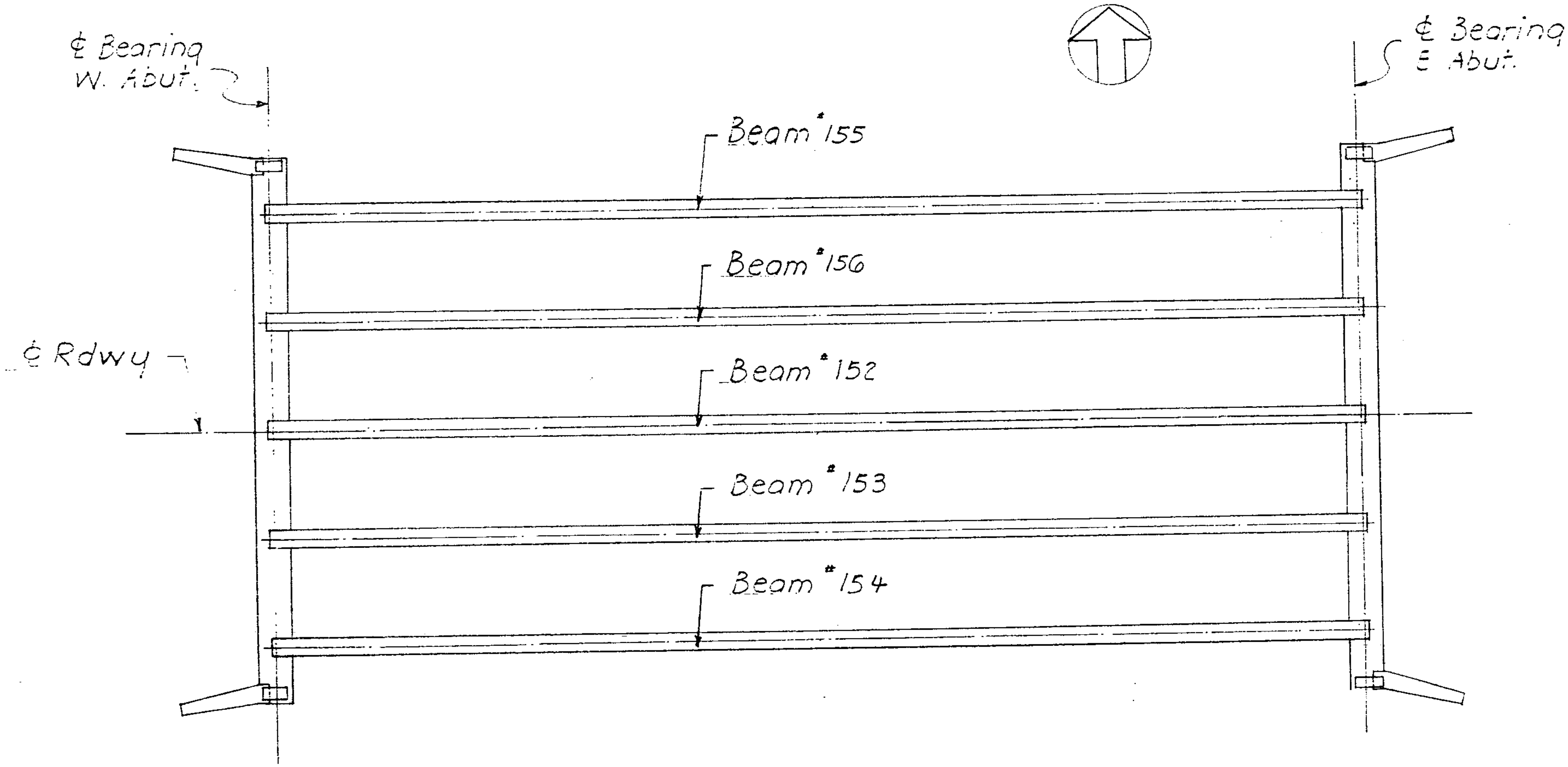


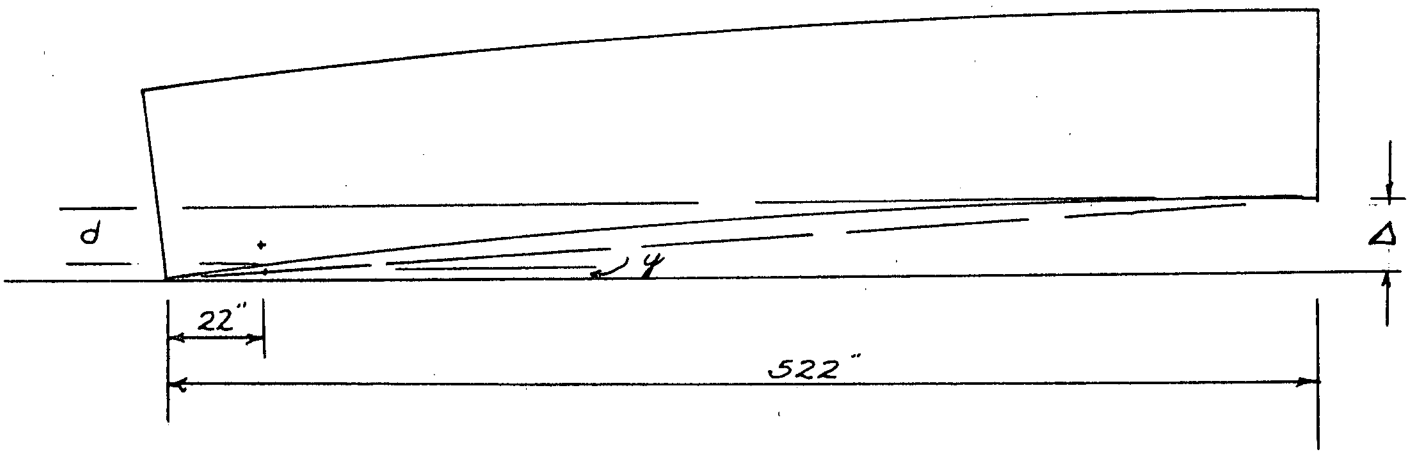
Figure 9, Beam Layout on Test Bridge

While the beams were in the storage yard all camber measurements were made by placing a level rod on the reference bolts and taking readings with a precise level. When the readings at each point had been taken it then becomes necessary to convert the displacement of the centerline to inches and add a correction to compensate for rotation of the end bolts about a point on the end of the beam. Figure 10 shows how the compensation is figured. The calculation of the correction involved one assumption which can be justified. It was necessary to assume that when the center of the beam deflected vertically upward, the reference bolts displaced vertically also, rather than on an arc about the rotation point which is the neutral axis. The rotation about the neutral axis, for a vertical deflection of 3.2" at the centerline, amounts to an angle of $0^{\circ}21'$.

When the assumption of vertical displacement of the end bolts is employed it is found the bolts will move vertically 0.13" when a deflection of 3.2" is observed at the center. The horizontal movement, X, would then be 0.0008 in, therefore, the assumption of vertical displacement is justified.

After the beams had been set on the abutments readings were taken on all reference points before any load was superimposed. When formwork for the deck was completed it became very difficult to read the rods by setting them on the top of the bolts. At this time, a single hole was drilled in the rods at the top so

Figure 10



In calculating the additional vertical displacement due to rotation, it is assumed that the bolt at the end moves vertically in direct proportion to the displacement at the centerline. The preceeding page served as a justification of this assumption.

The correction, y is given by:

$$y = \frac{22}{522} \Delta$$

Δ = Camber

Δ = y+d, where d is the rod reading

$$y = 0.0421\ (y+d)$$

$$y = \frac{0.0421}{0.9579}\ d$$

$$y = 0.0440\ d$$

they could be bolted to the centerline of the five beams and allowed to hang free for ease of reading. This method was employed to check camber measurements during the entire period of time the deck was being placed. Figures 11 and 12 show the set up during the deck pour. Readings were taken at short intervals of time and periodic checks were made on the bolts at the end of each beam to note the vertical displacement at these points.

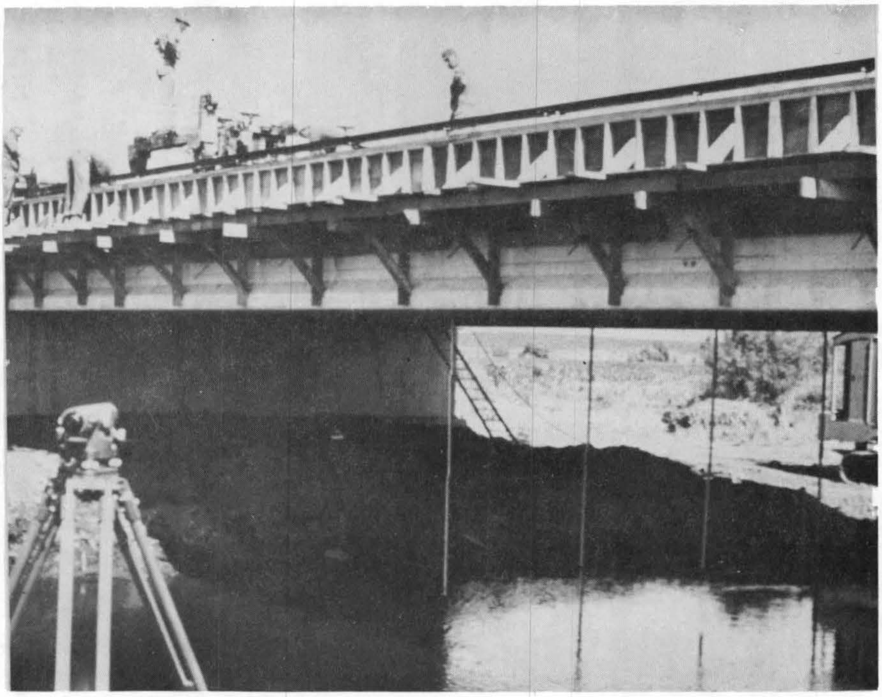


Figure 11, Set-up during deck placement

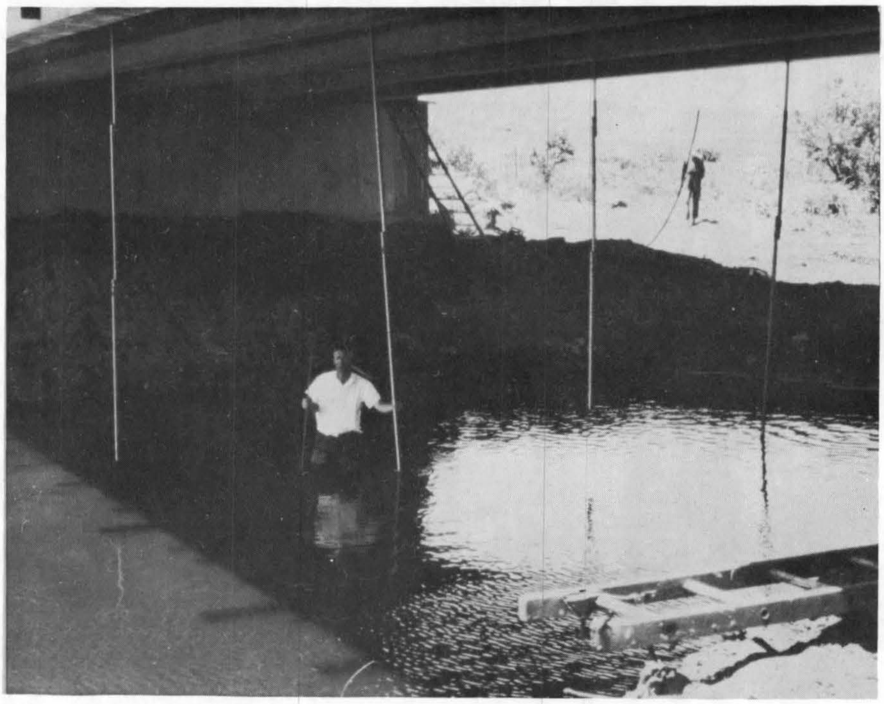


Figure 12, Procedure in reading rods

CHAPTER III

DISCUSSION OF RESULTS

3.1 Laboratory Tests

In addition to the cylinders which were cast at the time the beams were made, another series of cylinders were made to study the strength development pattern of the lightweight concrete. The cylinders were cured exactly as the beams; the first set was cured for 40 hours in steam, the second set 67 hours in steam. Table 4 shows the properties as they were determined. It should be noted that the f'c of the test cylinders cured for 67 hours averaged slightly less than that of the test cylinders cured for 40 hours.

Table 4, Compressive Strengths of Lightweight Concrete Cylinders

Date Cast	Age (days)	f'c (psi)	E_c [a] (psi $\times 10^6$)
4/15/68 [b]	7	5125	3.10
4/15/68	14	5560	3.23
4/15/68	28	5980	3.35
4/15/68	58	6360	3.46
4/19/68 [c]	7	4915	3.04
4/19/68	14	5570	3.23
4/19/68	54	5925	3.34

a. Computed by $E_c = 33\sqrt{w^3 \cdot f'_c}$ where w has an average value of 120 lb/ft^3.

b. 40 hours steam cure.

c. 67 hours steam cure.

3.2 Camber Development

Figures 13 to 17 indicate the pattern of the camber development from the time when the prestress was transferred to the beams until about 200 days after the deck was placed. Figure 18 shows the development of camber in a typical beam. The deck was cast with normal weight sand and gravel concrete.

There was a slight difference in the camber development between the two sets of beams. Group I, consisting of beams 152, 153, and 154, had developed a larger value of camber, prior to deck placement, than the Group II beams. The beams in Group I were about 0.1" below the value of 3.20" which had been predicted for them. The beams in Group II had a value which was approximately 0.3" below the predicted value of 3.20". The design calculation for the beams and the camber predictions are shown on pages A-4 and A-5 in the Appendix.

During the time the deck was being placed continuous readings were taken on the reference bolts as described in Chapter II, section 4. The graphs in figures 19 to 23 show how the camber of the beam varied during the deck pour.

The pour started at the west end of the bridge and proceeded east across the bridge. The initial set of readings was taken with the finish machine "in place" on the west end of the bridge. The apparent rebound near the end could be attributed to the

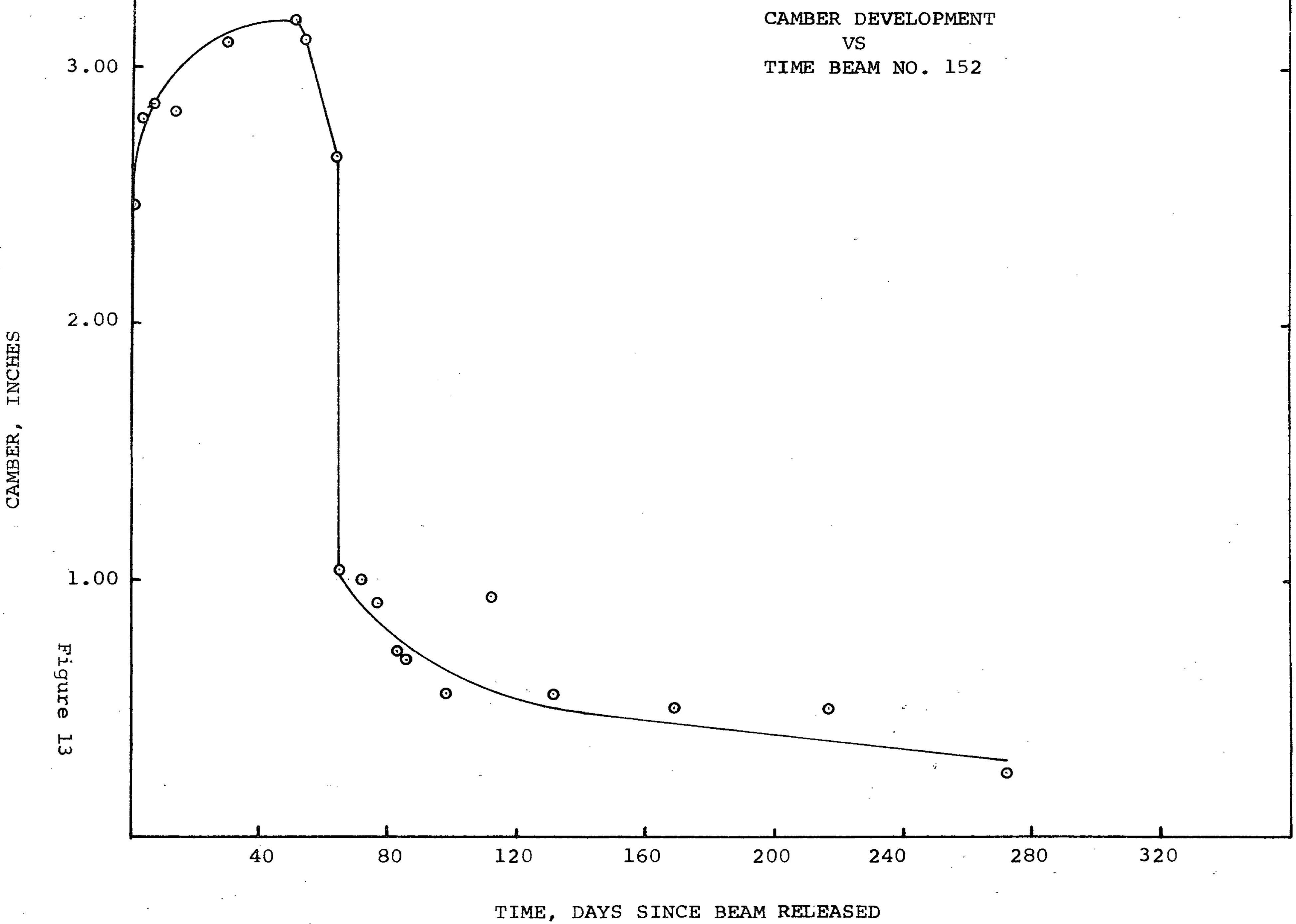

CAMBER DEVELOPMENT
VS
TIME BEAM NO. 152
CAMBER, INCHES
3.00
2.00
1.00
40
80
120
160
200
240
280
320
TIME, DAYS SINCE BEAM RELEASED

Figure 13

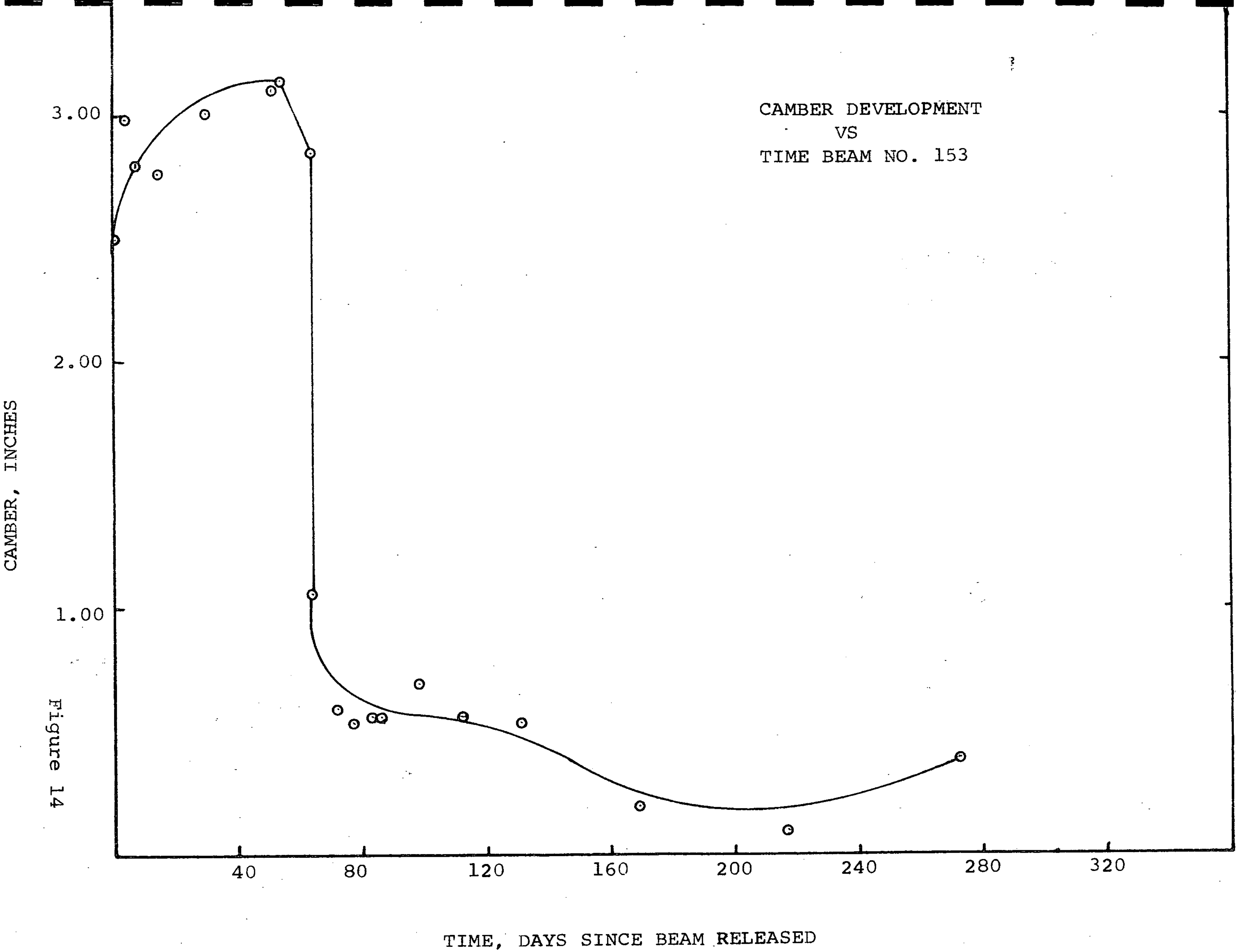

CAMBER DEVELOPMENT
VS
TIME BEAM NO. 153
3.00
2.00
1.00
CAMBER, INCHES
40
80
120
160
200
240
280
320
TIME, DAYS SINCE BEAM RELEASED

Figure 14

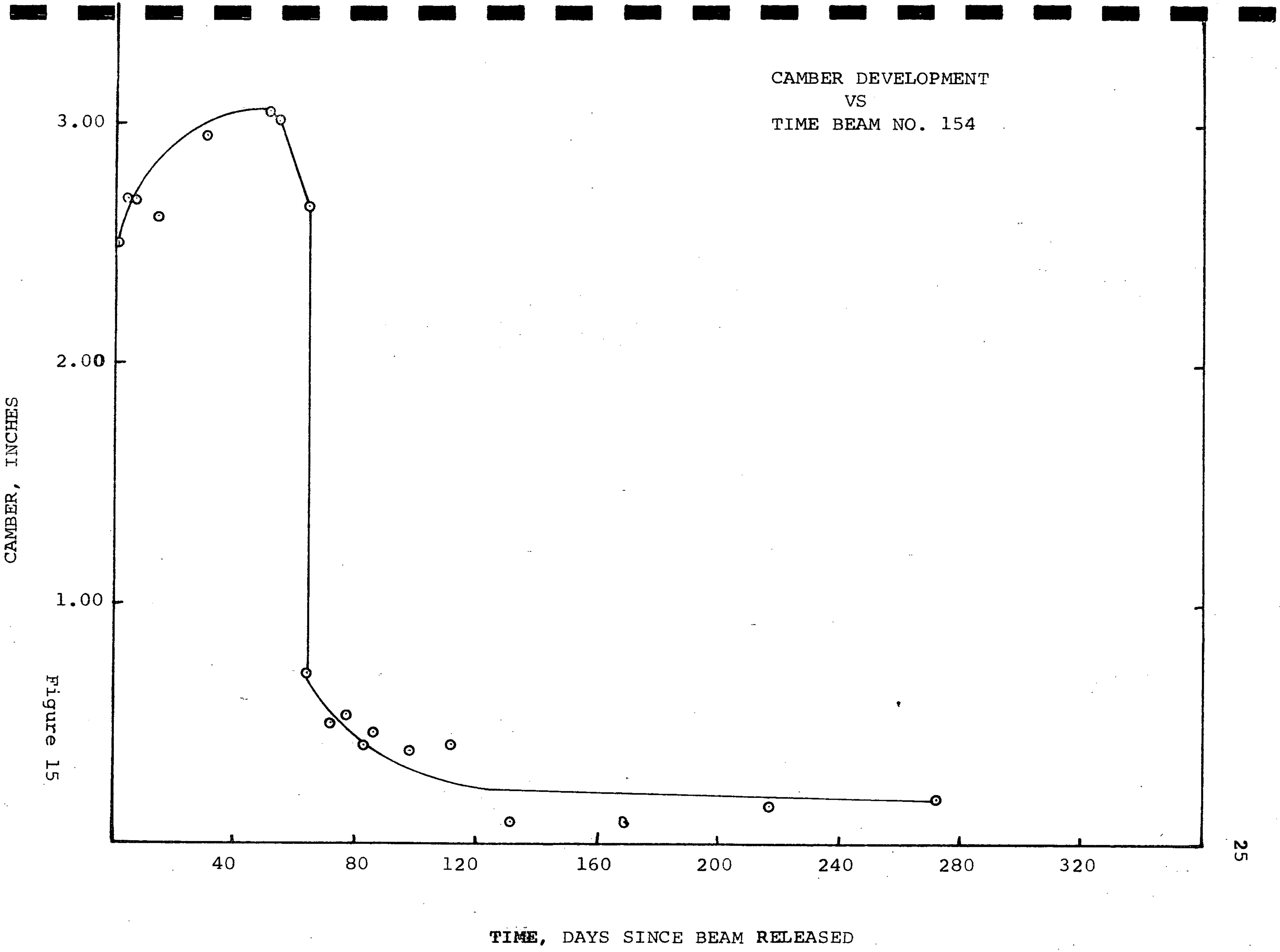

CAMBER DEVELOPMENT
VS
TIME BEAM NO. 154
3.00
2.00
1.00
CAMBER, INCHES
40
80
120
160
200
240
280
320
TIME, DAYS SINCE BEAM RELEASED

Figure 15

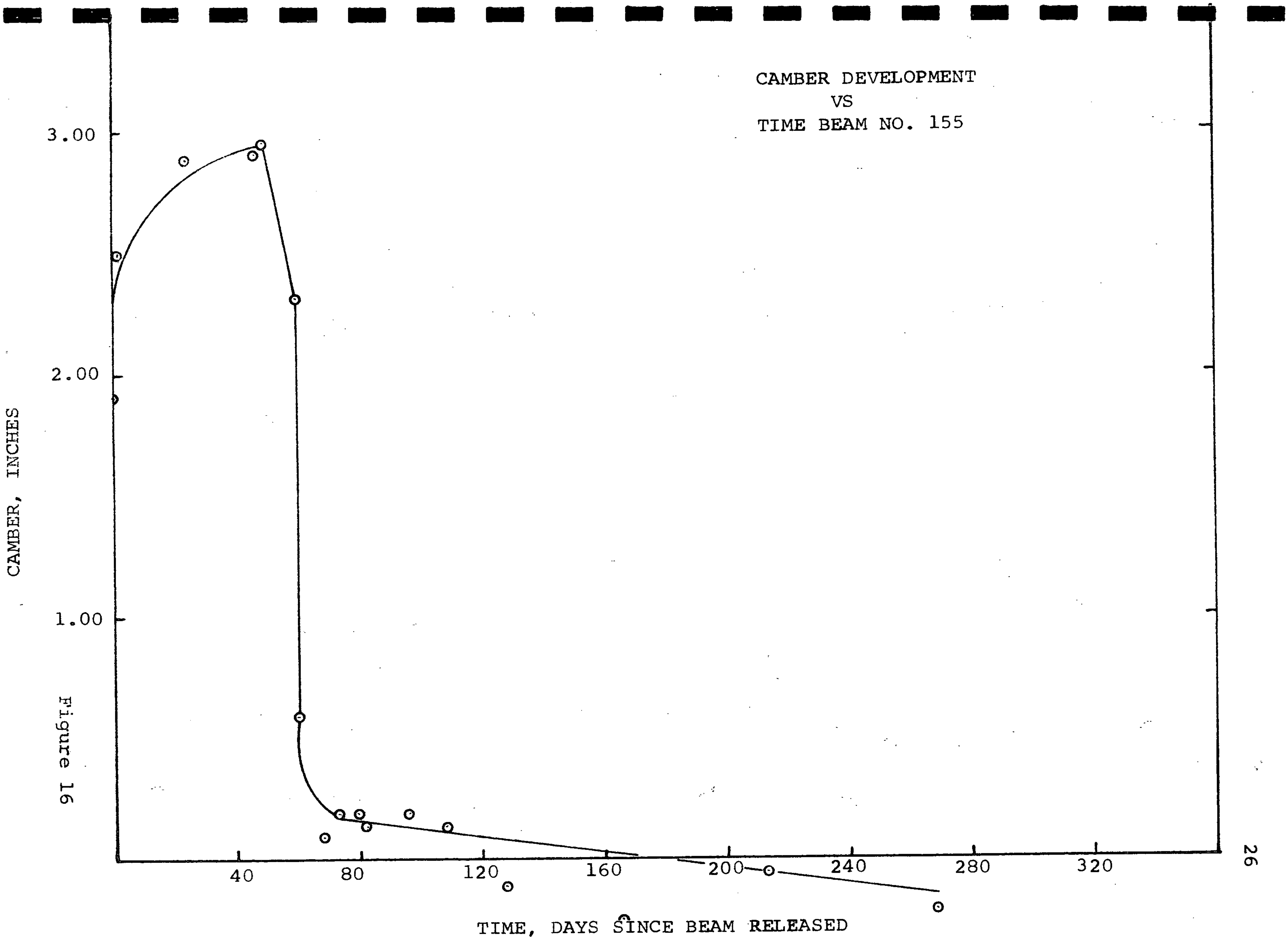

CAMBER DEVELOPMENT
VS
TIME BEAM NO. 155
3.00
2.00
1.00
CAMBER, INCHES
40
80
120
160
200
240
280
320
TIME, DAYS SINCE BEAM RELEASED

Figure 16

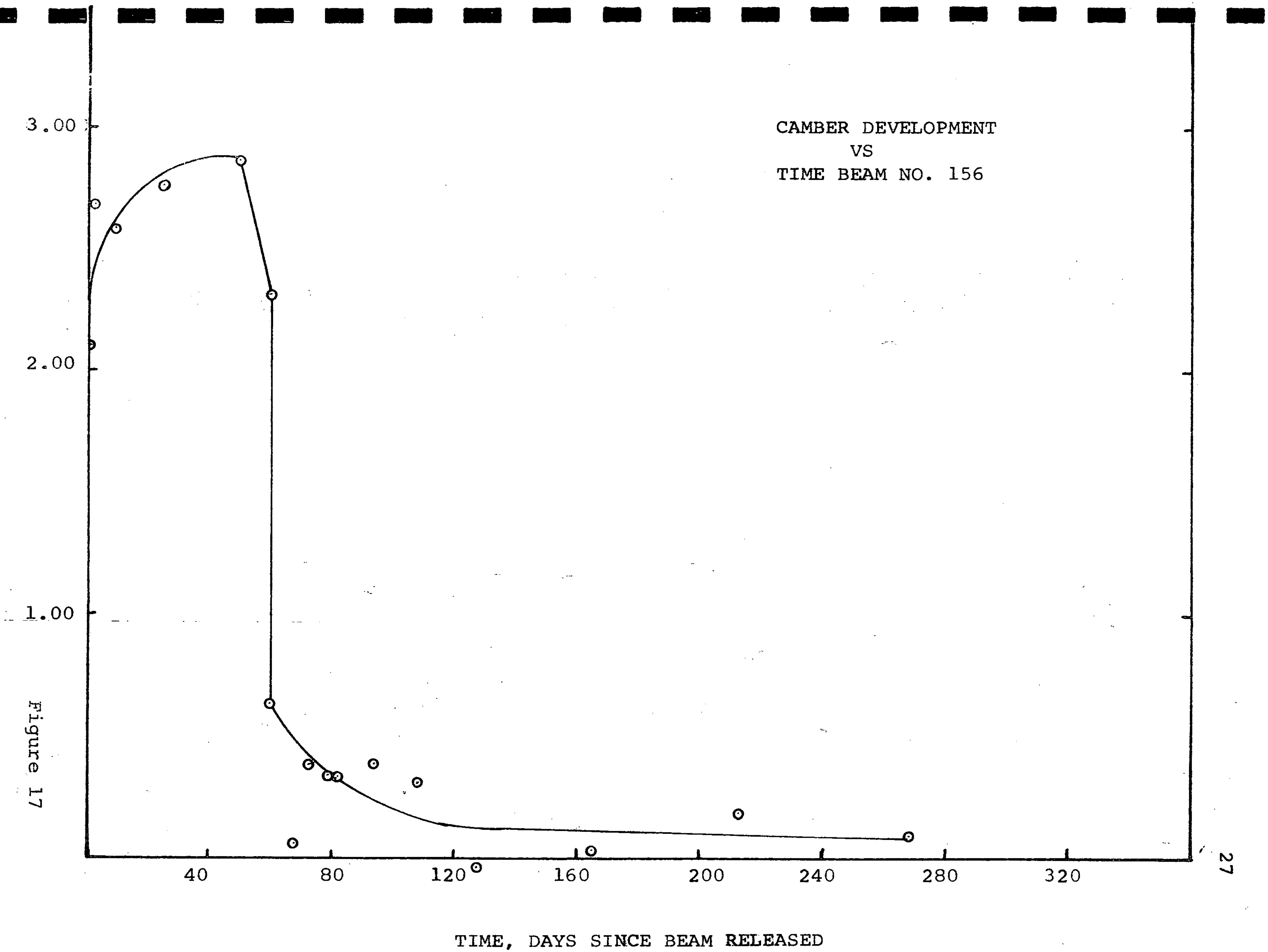

CAMBER DEVELOPMENT
VS
TIME BEAM NO. 156
3.00
2.00
1.00
40
80
120
160
200
240
280
320
TIME, DAYS SINCE BEAM RELEASED

Figure 17

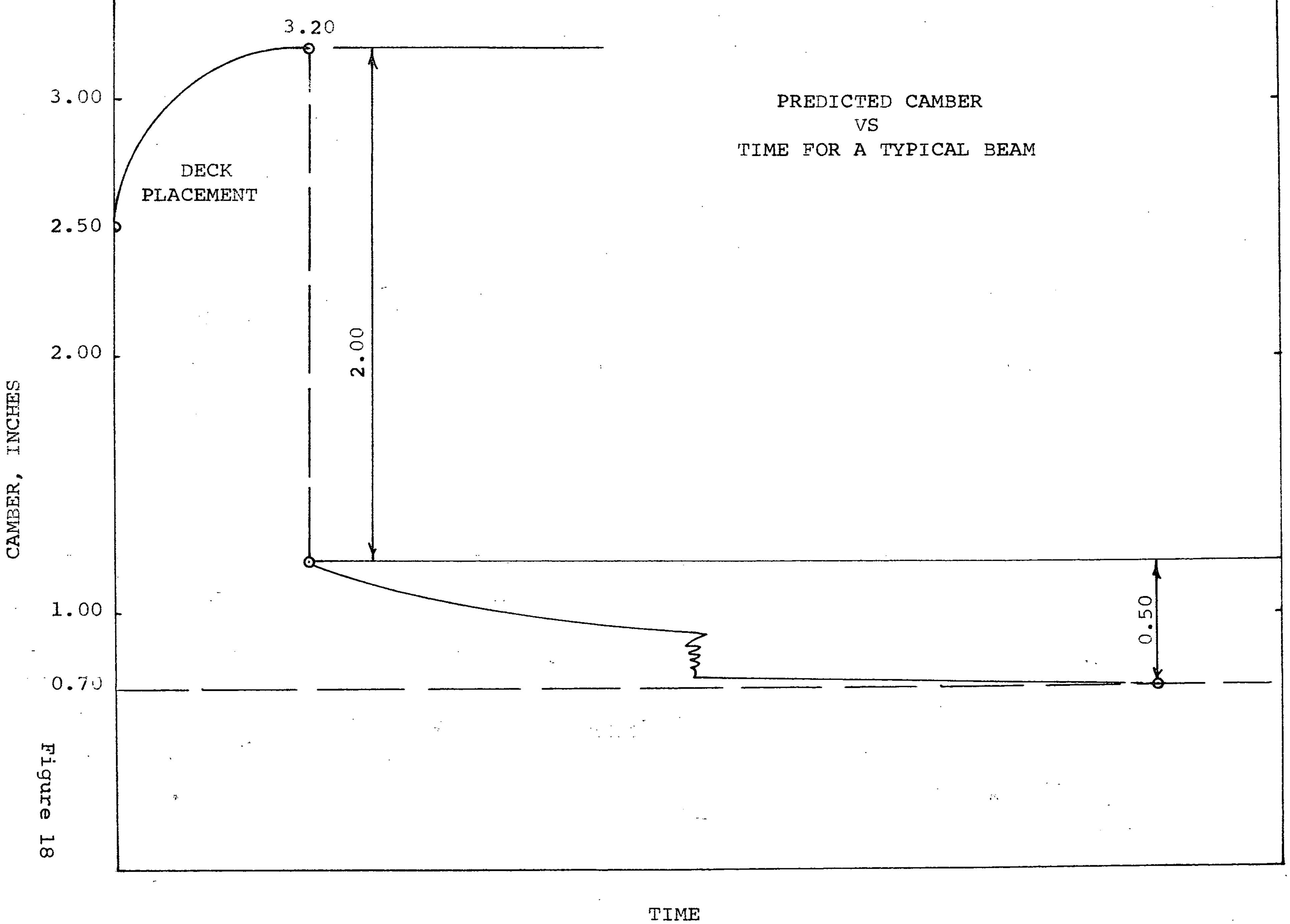

PREDICTED CAMBER
VS
TIME FOR A TYPICAL BEAM
3.20
3.00
2.50
2.00
1.00
0.70
DECK
PLACEMENT
2.00
0.50
CAMBER, INCHES
TIME

Figure 18

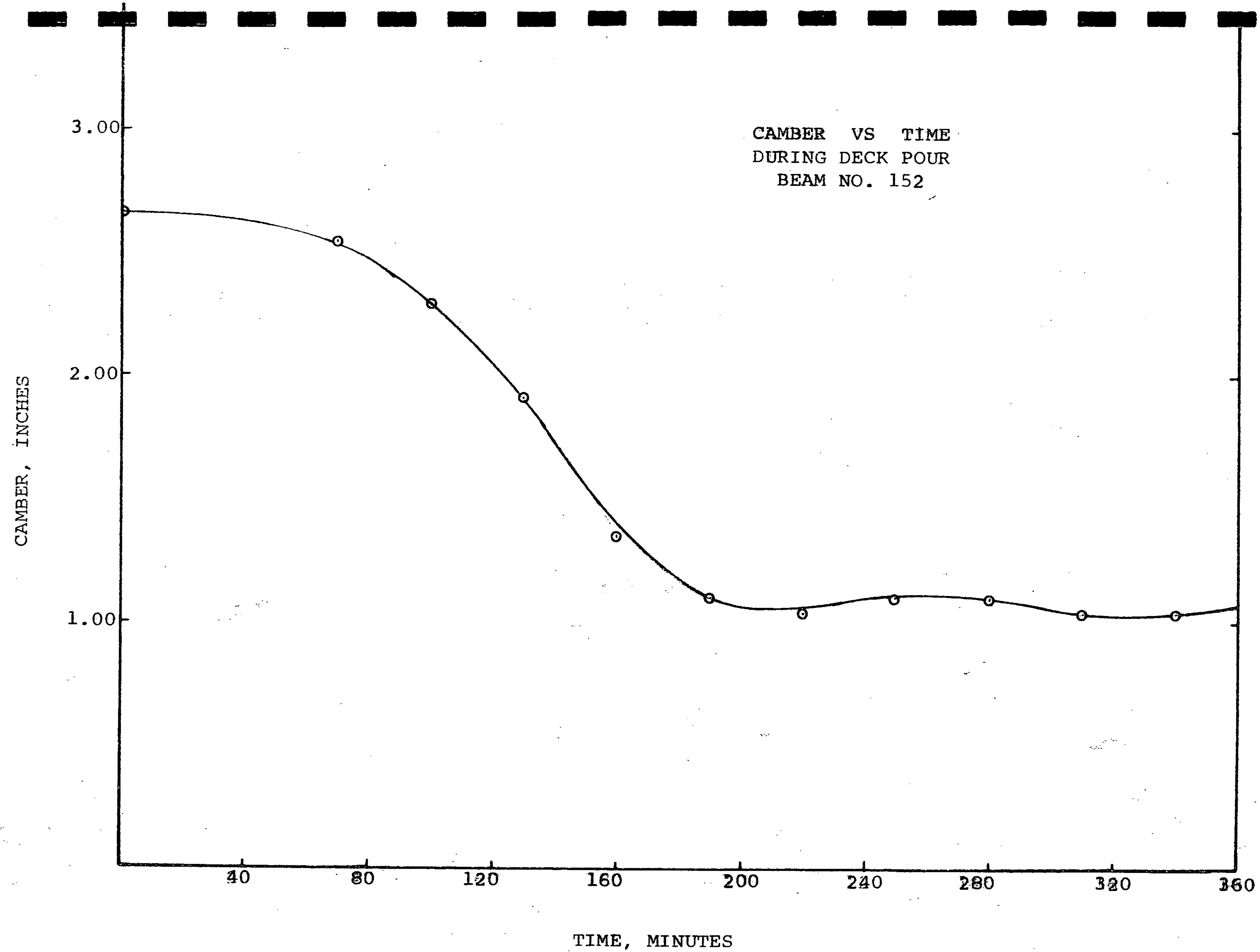

CAMBER VS TIME
DURING DECK POUR
BEAM NO. 152
3.00
2.00
1.00
CAMBER, INCHES
40
80
120
160
200
240
280
320
360
TIME, MINUTES

Figure 19

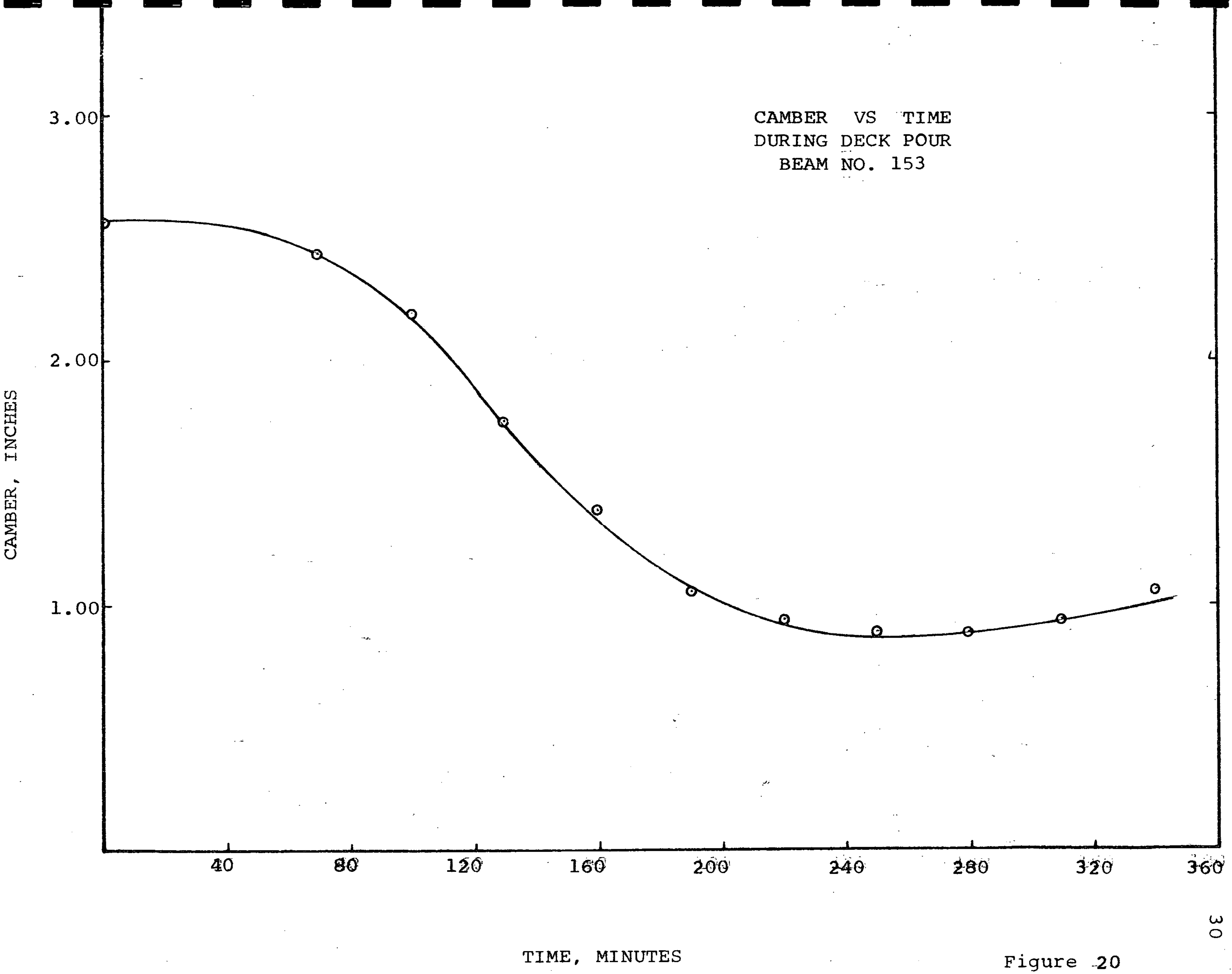

CAMBER VS TIME
DURING DECK POUR
BEAM NO. 153
3.00
2.00
1.00
CAMBER, INCHES
40
80
120
160
200
240
280
320
360
TIME, MINUTES

Figure 20

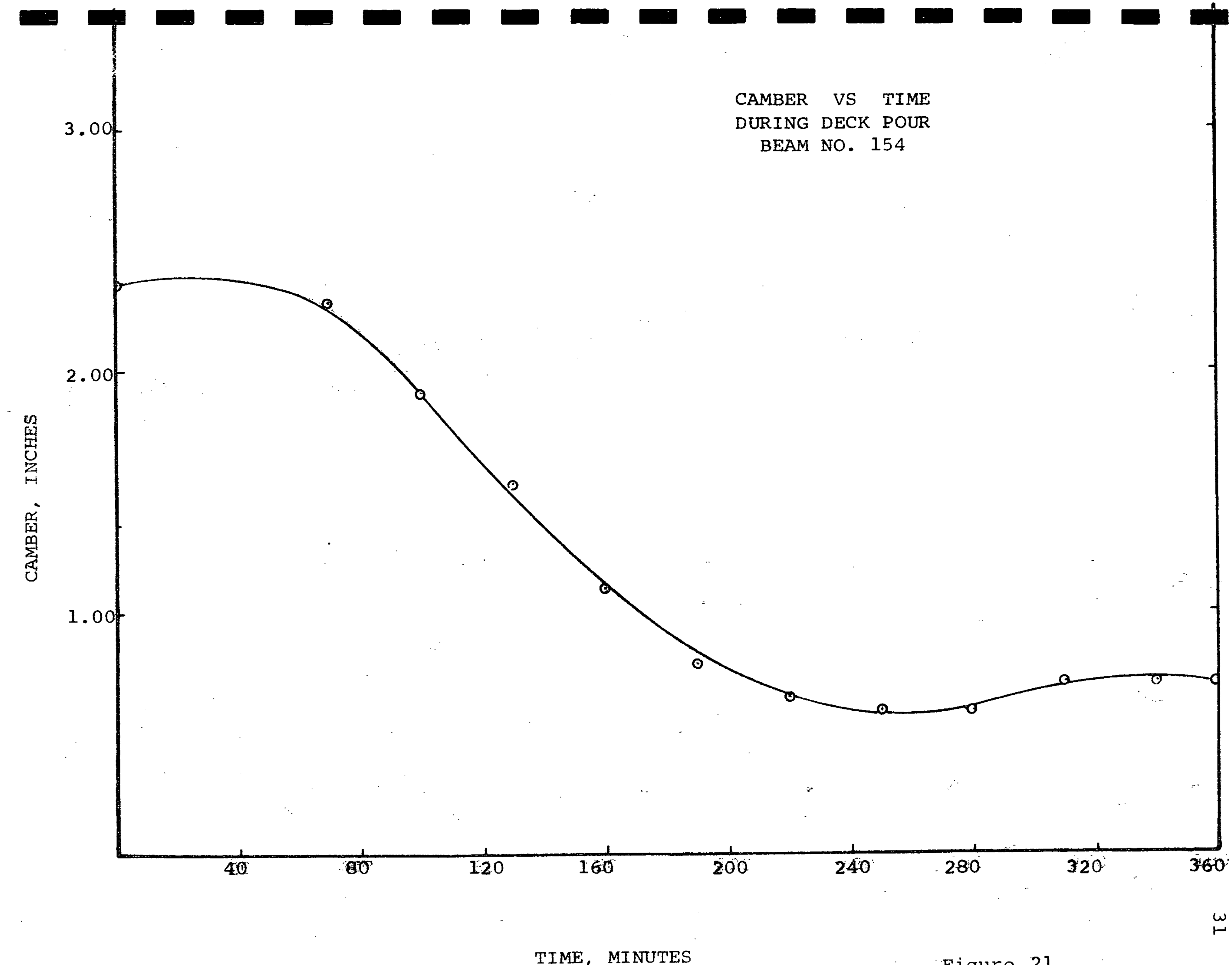

CAMBER VS TIME
DURING DECK POUR
BEAM NO. 154
3.00
2.00
1.00
CAMBER, INCHES
40
80
120
160
200
240
280
320
360
TIME, MINUTES

Figure 21

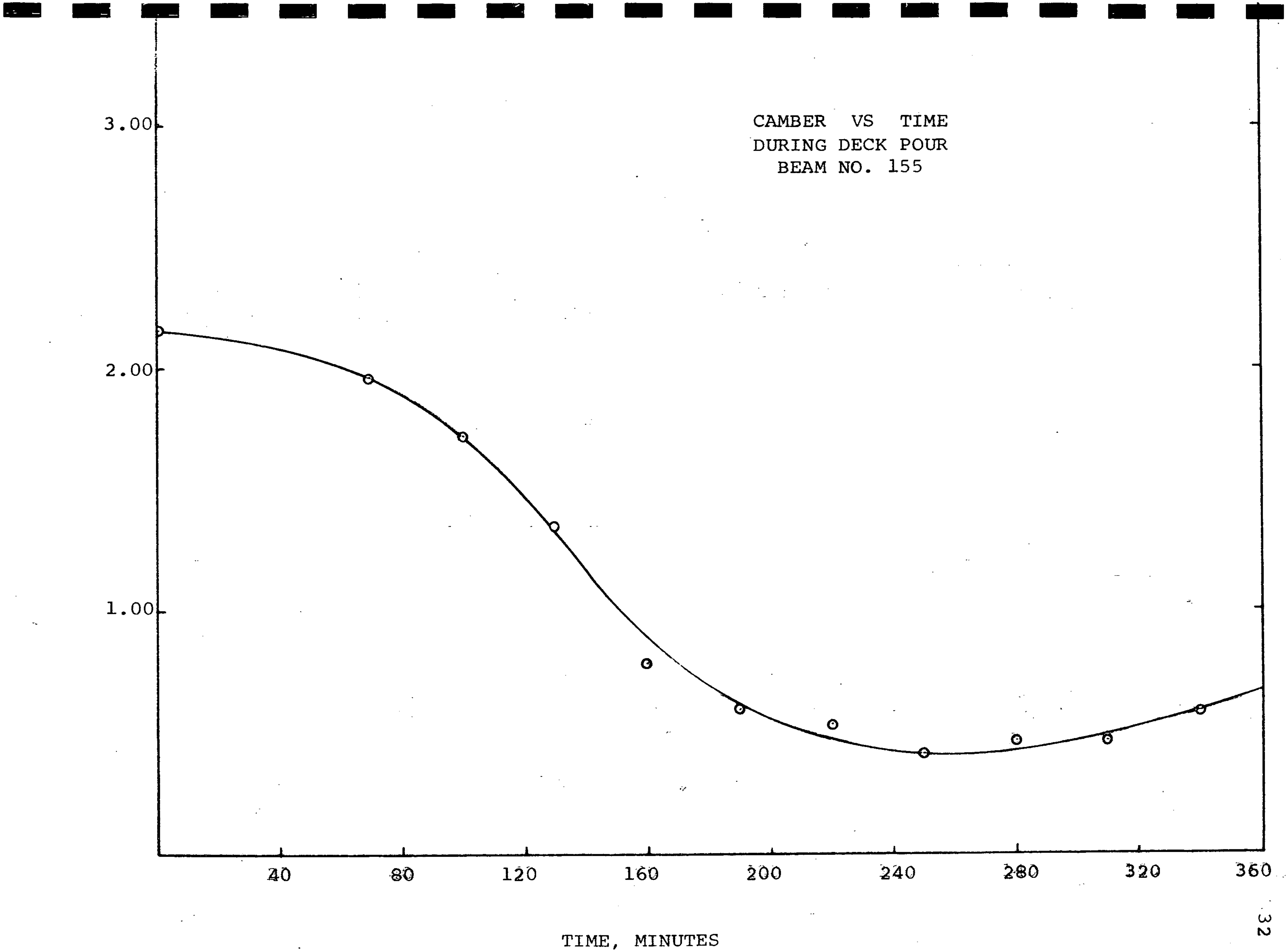

CAMBER VS TIME
DURING DECK POUR
BEAM NO. 155
3.00
2.00
1.00
40
80
120
160
200
240
280
320
360
TIME, MINUTES

Figure 22

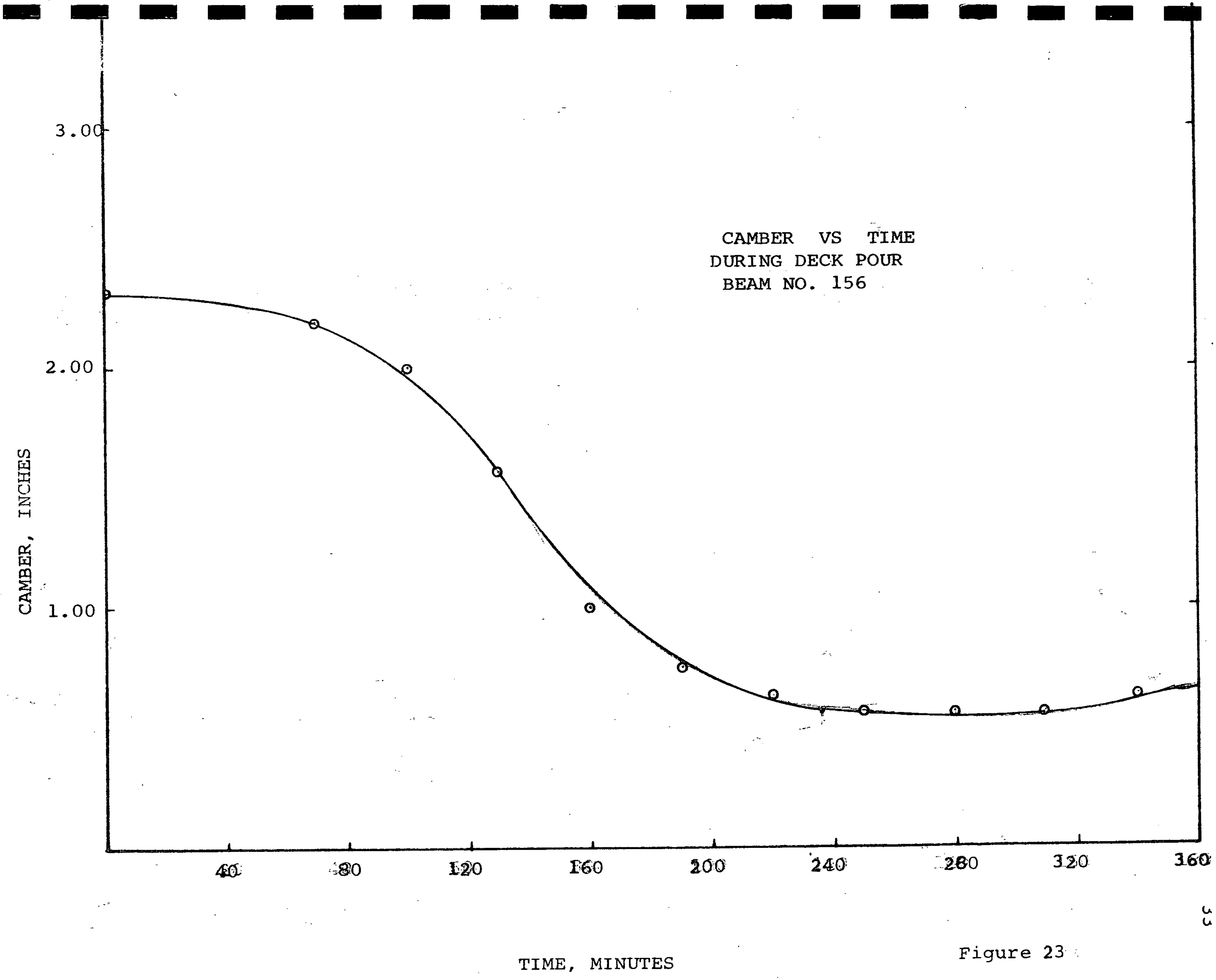

CAMBER VS TIME
DURING DECK POUR
BEAM NO. 156
3.00
2.00
1.00
CAMBER, INCHES
40
80
120
160
200
240
280
320
360
TIME, MINUTES

Figure 23

finish machine being removed at about the 280 minute mark and the last 2 readings on each beam taken without the finish machine on the deck.

No cylinders for the Group II beams were broken at the 28 day mark, however we can get a comparison of the strengths and calculated modulus of elasticity for Group I and Group II, at 58 and 54 days respectively. The 435 psi difference in f'_c and 0.12×10^6 psi difference in modulus of elasticity are not significant to where this data could be used to explain the difference in camber between Group I and Group II.

A factor which would have caused a difference in the camber was the different conditions under which the beams were cured. The time spent in raising the steam to curing levels was different for the two groups. It took nearly 4 hours to raise the steam on Group I and only 2½ hours to raise the steam on Group II. Group I was steam cured 40 hours while Group II was cured 67 hours. These conditions could have had an effect on the creep characteristics and could change the pattern of camber development completely. There are many inherent factors in concrete, especially in lightweight concrete, by which the camber is affected. Aggregate type and shrinkage characteristics are just two more factors which could affect the camber development.

The effect of the deck placement is to rapidly decrease the camber under the action of the dead load (deck). Initially it was estimated that 3.20" of camber would be in the beam at the time the beams were to be set. It was predicted that the dead-load would cause the beams to deflect about 2.00" thus leaving approximately 1.20" of camber in the beam after the dead load was applied. Creep and shrinkage will cause the beam to deflect approximately 0.50" over a period of time.

Chapter IV

Observations

In looking at the results, it is evident that the work done under Iowa Highway Research Board, Project HR-104 is consistent with work done by others.[1] Camber (inches) vs time (days after release) has indicated that the results of this work are fairly consistent with the predicted values.

The method of measuring camber that was used was rather simple yet it afforded very good results. The following table compares the predicted results with the measured results of various stages in the project development.

Table 5, COMPARISON OF PREDICTED AND MEASURED CAMBER AND DEFLECTION VALUES

	Initial Camber		Final Camber(a)		Slab D.L. Deflection		Deflection after Slab in Place	
Beam No.	Pred.	Meas.	Pred.	Meas.	Pred.	Meas.	Pred.	Meas.
152	3.20	3.10	0.70	0.25	2.00	2.10	1.20	1.05
153	3.20	3.15	0.70	0.40	2.00	2.10	1.20	1.05
154	3.20	3.00	0.70	0.20	2.00	2.30	1.20	0.70
155	3.20	3.00	0.70	0.20	2.00	2.35	1.20	0.60
156	3.20	2.85	0.70	0.10	2.00	2.25	1.20	0.65

a) as of January 14, 1969

The initial values of camber appeared to be slightly lower than what was predicted. The dead load deflections were somewhat larger than the 2.00 inches which was initially calculated. It appears the creep and shrinkage varied somewhat from their predicted value of 0.5",

ACKNOWLEDGEMENTS

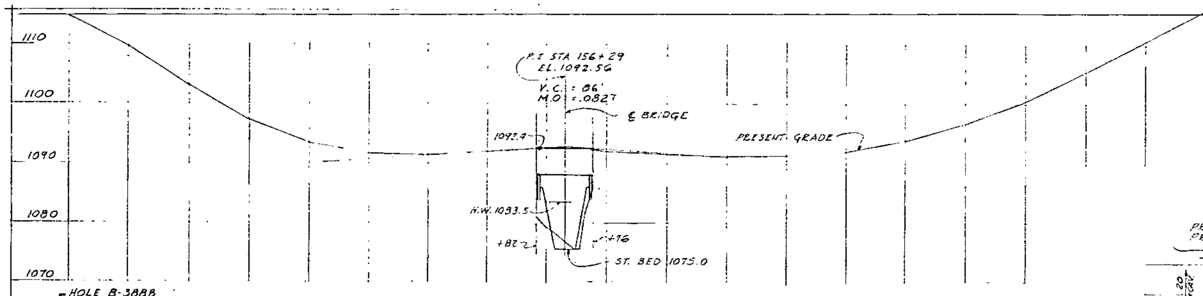
The author wishes to thank Mr. Charles Pestotnik, Bridge Engineer, Iowa State Highway Commission, Mr. Henry Gee, Assistant Bridge Engineer, Iowa State Highway Commission and Mr. James Boehmler, Jr., President of Prestressed Concrete of Iowa, Inc., Iowa Falls, Iowa; without the aid of these men the preparation of this report would have been very difficult.

REFERENCES

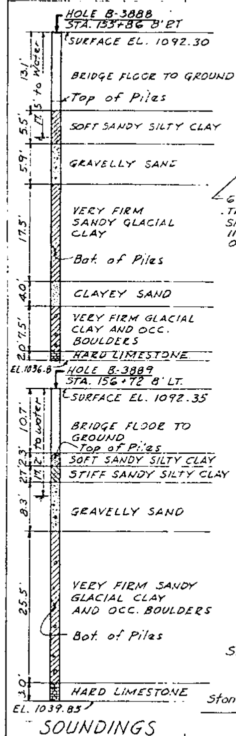
1. Furr and Sinno, "Creep in Prestressed Lightweight Concrete", Texas Transportation Institute, October, 1967.

APPENDIX

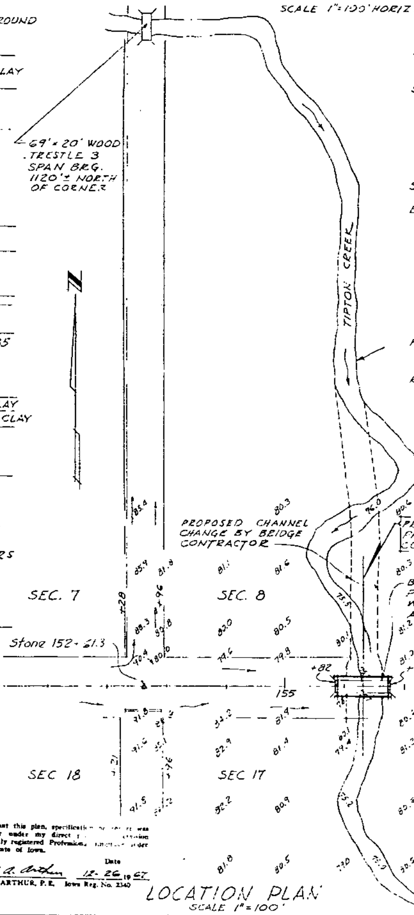
BENCH MARK: SPIKE IN WING PILE AT N.W. CORNER BRIDGE EL. 1092.35



PROFILE
SCALE 1"=100' HORIZ. 1"=10' VERT.



SOUNDINGS
DATE 10-31-67



PROPOSED 86' × 32'
PRETENSIONED PRESTRESSED
CONCRETE BRIDGE @ STA. 156+29

BRIDGE CONTRACTOR TO REMOVE
PRESENT 3 SPAN 94' × 20' I-BM. BRIDGE
WITH WOOD FLOOR & WOOD SUBSTRUCTURE
AT STA. 156+24

LOCATION PLAN
SCALE 1"=100'

GENERAL NOTES

DESIGN:
THIS BRIDGE IS DESIGNED FOR H20-44 LOADING PLUS AN ALLOWANCE OF 19 LBS PER SQ. FT. FOR FUTURE WEARING SURFACE.

SPECIFICATIONS:
DESIGN: A.A.S.H.O. 1965.
CONSTRUCTION: STANDARD SPECIFICATIONS OF THE IOWA STATE HIGHWAY COMMISSION SERIES OF 1964 AND CURRENT APPLICABLE SPECIAL PROVISIONS AND SUPPLEMENTAL SPECIFICATIONS.
CONCRETE FOR PRETENSIONED PRESTRESSED BEAMS TO BE IN ACCORDANCE WITH IOWA STATE HIGHWAY COMMISSION SPECIAL PROVISIONS FOR LIGHT WEIGHT CONCRETE BRIDGE UNITS, NO. 624, DATED MARCH 6, 1968.

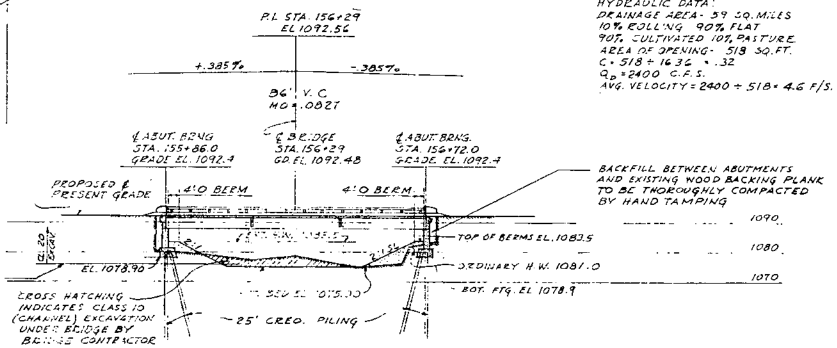
STANDARD REFERENCES:
ALUMINUM HANDRAIL DETAILS: STANDARD V14-34-64 DATED AUGUST 1967, REVISED 10-10-67.

EARTH WORK:
BRIDGE CONTRACTOR TO MAKE CHANNEL CHANGE AS INDICATED WASTING EXCAVATED MATERIAL ALONG BANKS AS DIRECTED BY THE ENGINEER. HE SHALL CLEAR CHANNEL UNDER BRIDGE (APP. 280 CU.YDS. CLASS 10 CHANNEL EXCAV.) TO LINES AND LEVELS SHOWN USING EXCAVATED MATERIAL TO BUILD, SHAPE AND COMPACT BERMS AND BACKFILL BEHIND ABUTMENTS. COST OF BUILDING BERMS AND BACKFILLING BEHIND ABUTMENTS TO BE INCLUDED IN PRICE BID FOR CLASS 10 EXCAVATION.
BRIDGE CONTRACTOR SHALL WIDEN PRESENT ROADWAY SHOULDERS AT BRIDGE AND BACKFILL BEHIND WINGS TAPERING SHOULDER LINES INTO PRESENT SHOULDERS ABOUT 25' BACK OF ENDS OF WINGS, USING CLASS 24 EXCAVATION OBTAINED WITHIN 1500 FT. OF SITE, AS DIRECTED BY THE ENGINEER.

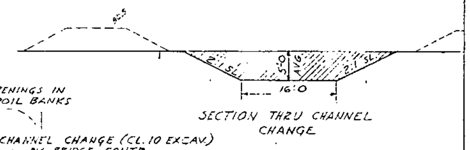
PILE DRIVING:
ALL ABUTMENT PILE SHALL BE DRIVEN TO FULL PENETRATION IF PRACTICABLE BUT TO NOT LESS THAN 22 TONS BEARING VALUE.

REMOVAL OF PRESENT STRUCTURE:
BRIDGE CONTRACTOR SHALL REMOVE PRESENT 3 SPAN STEEL I-BM. BRIDGE WITH WOOD FLOOR AND WOOD PILE PIERS AND ABUTMENTS IN ACCORDANCE WITH SECTION 2401 OF THE SPEC'S. EXCEPT NO MATCHMARKING OF STEEL WILL BE REQUIRED. STEEL BEAMS ARE TO BE CUT INTO THREE APROX. EQUAL LENGTHS AND ALL MATERIAL, INCLUDING STRUCTURAL STEEL BEAMS AND MEMBERS, WOOD PLANKS WITH NAILS REMOVED AND PIER PILE TO BE STACKED NEATLY NEAR SITE AS DIRECTED BY THE ENGINEER. EXISTING PIER PILING TO BE CUT OFF 1'-0 BELOW STREAM BED AND ONLY THOSE PORTIONS OF WOOD ABUTMENTS INTERFERING WITH NEW CONSTRUCTION OR LESS THAN 1'-6 BELOW GRADE SHALL BE REMOVED

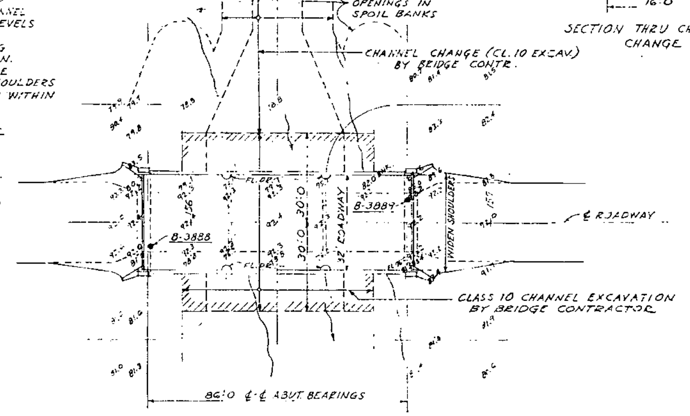
HYDRAULIC DATA:
DRAINAGE AREA - 59 SQ. MILES
10% ROLLING 90% FLAT
90% CULTIVATED 10% PASTURE
AREA OF OPENING - 518 SQ. FT.
C = 518 ÷ 1636 = .32
Q_D = 2400 C.F.S.
AVG. VELOCITY = 2400 ÷ 518 = 4.6 F/S.



LONGITUDINAL SECTION ON ℄ ROADWAY
SCALE 1"=20'-0



SECTION THRU CHANNEL CHANGE



SITUATION PLAN
SCALE 1"=20'

ESTIMATED QUANTITIES

PART	PRETENSIONED PRESTRESSED CONC. BEAMS	STRUCTURAL CONCRETE CU. YDS.	REINFORCING STEEL LBS.	ALUMINUM RAIL LIN. FT.	CREO. WOOD PILING LIN. FT.	EXCAVATION - CU. YDS.			GRANULAR BACKFILL CU. YDS.	REMOVAL OF PRESENT BRIDGE
						CLASS 20	CLASS 10 CHANNEL	CLASS 24 (BORROW)		
SUPERSTRUCTURE	5@86' C-SPEC.	90.0	22452	169.67						
ABUTMENTS		94.2	9475		36@25'	105				
TOTAL	5	* 184.2	31927	169.67	900	105	* 1594	150	40	LUMP SUM

* ABUT. END POSTS (.9 CU. YDS.) AND SUPERSTRUCTURE FLOOR, CURBS AND DIAPHRAGMS (90 CU. YDS.) TO BE CLASS D CONCRETE. REMAINDER OF CONCRETE (93.3 CU. YDS.) TO BE CLASS C

* 1314 CU. YDS. CHANNEL CHANGE
* 280 " " CHANNEL CLEARANCE UNDER BRIDGE

I hereby certify that this plan, specification or report was prepared by me or under my direct personal supervision and that I am a duly registered Professional Engineer under the laws of the State of Iowa.
Signed Herbert A. Arthur Date 12-26 1967
HERBERT A. ARTHUR, P.E. Iowa Reg. No. 2340

LOCATION:
SECTION 17
TIPTON TWP.
T87N R21W
OVER TIPTON CREEK
HARDIN COUNTY
IOWA

SINGLE SPAN 86' × 32' PRETENSIONED PRESTRESSED CONCRETE BRIDGE

ALUMINUM RAIL — CONCRETE ABUTS.

GENERAL DRAWING

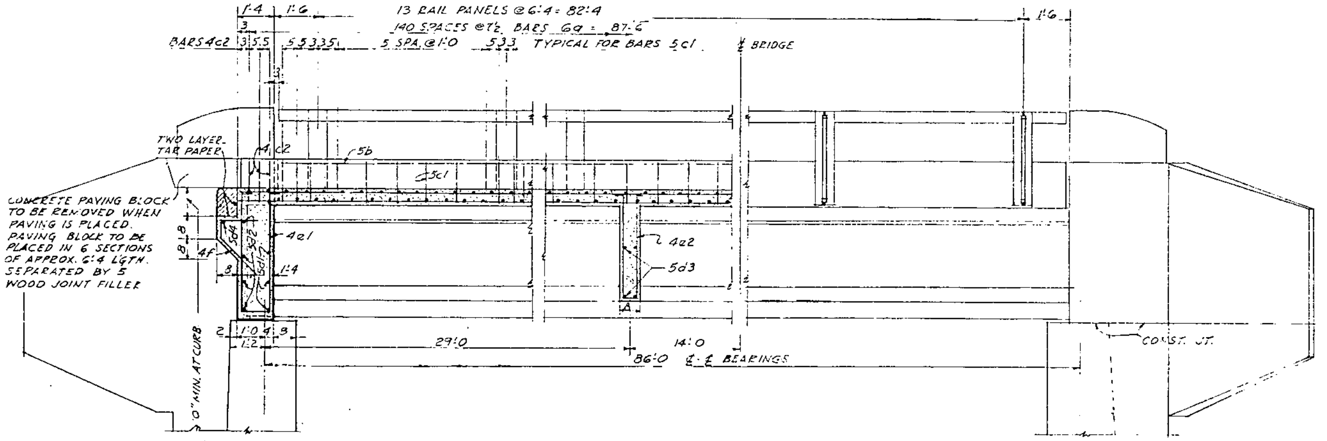
STATION 156+29 — COUNTY PROJECT LFM-1886(2)--73-42

HARDIN COUNTY

HERBERT A. ARTHUR - CONSULTING ARCHITECT-ENGINEER
P. F. BARNARD - CONSULTING STRUCTURAL ENGINEER
AMES IOWA

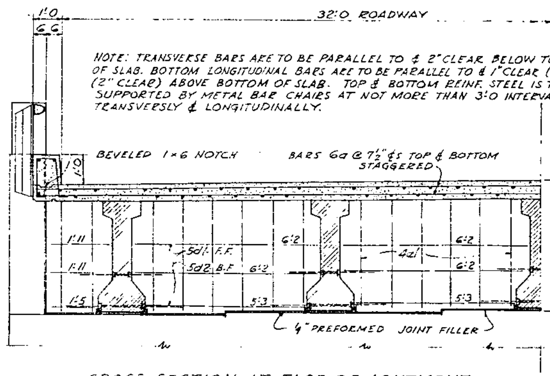
NOVEMBER 1967 — SHEET 2 OF 5

DESIGN NO. 168 — HARDIN COUNTY — FILE NO. 23087

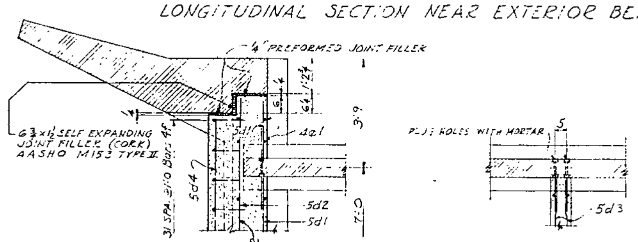


LONGITUDINAL SECTION NEAR EXTERIOR BEAM

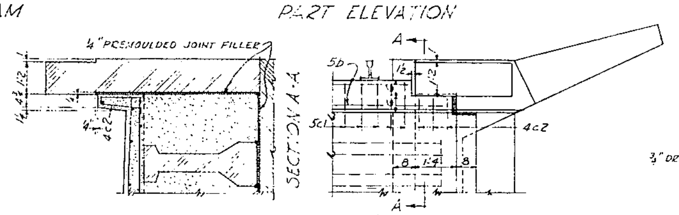
PART ELEVATION



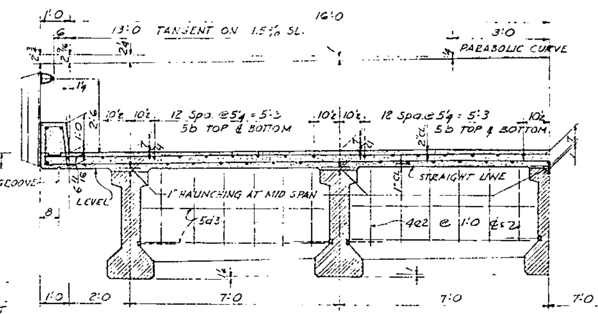
CROSS SECTION AT FACE OF ABUTMENT



SECTION BELOW SLAB



PLAN VIEW AT ABUTMENT



CROSS SECTION AT MID-SPAN

GENERAL NOTES:

THIS BRIDGE IS DESIGNED FOR H20-44 LOADING PLUS 19 LBS/SQ.FT. FUTURE WEARING SURFACE. TOP ½" OF SLAB IS CONSIDERED WEARING SURFACE.

ALL EXPOSED CORNERS 90° OR SHARPER ARE TO BE FORMED WITH ¾" DRESSED FILLET STRIP.

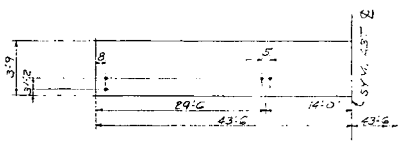
CLEAR DISTANCE FROM FACE OF CONCRETE TO ANY BAR 1½" UNLESS OTHERWISE SHOWN. ALL REINFORCING BARS TO BE SECURELY WIRED IN PLACE AND ADEQUATELY SUPPORTED ON METAL BAR CHAIRS BEFORE CONCRETE IS PLACED.

ALL BEAMS ARE TO BE SET VERTICAL.

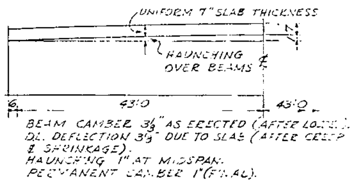
FORMS FOR THE SLAB AND CURB ARE TO BE SUPPORTED ON THE PRESTRESSED BEAMS.

SLAB CONCRETE IS TO HAVE A 28 DAY CRUSHING STRENGTH OF 3500 P.S.I. AND CONTAIN NO CLASS "V" AGGREGATE. IT IS TO BE PLACED AS DRY AS PRACTICABLE TO REDUCE SHRINKAGE TO A MINIMUM. SPECIAL PRECAUTIONS ARE TO BE TAKEN TO SECURE COMPLETE BOND BETWEEN SLAB AND BEAM.

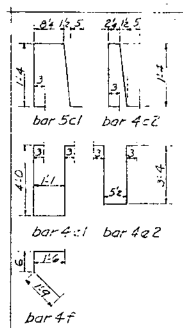
COST OF ALL JOINT MATERIAL AND DRAINS TO BE INCLUDED IN PRICE BID FOR CONCRETE. COST OF BEARING MATERIAL FOR BEAMS TO BE INCLUDED IN PRICE BID FOR PRESTRESSED CONCRETE BEAMS.



LOCATION OF COIL TIES



CAMBER & DEFLECTION (ANTICIPATED)

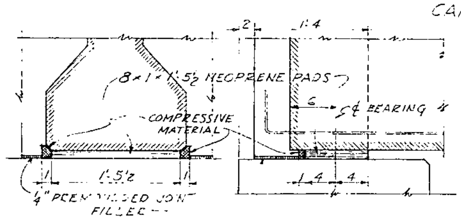


BAR LIST FOR SUPERSTRUCTURE

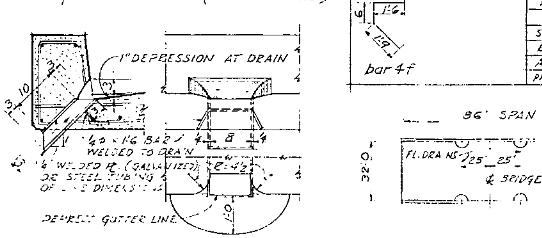
	LOCATION	SHAPE	NO.	LGTH	WEIGHT
6a	SLAB TRANSVERSE - TOP & BOTTOM	—	273	32-9	14143
5b	SLAB & CURB LONGIT. " " "	—	128	44-11	5996
5c1	CURB VERTICAL	⊓	220	3-8	841
4c2	CURB VERTICAL AT ENDS	⊓	12	3-2	26
5d1	END DIAPHRAGM LONGIT. F.F.	—	36	Var.	169
5d2	" " " B.F.	—	8	32-8	273
5d3	INTERIOR DIAPHRAGM LONGIT.	—	32	6-2	206
4a1	STIRRUPS - END DIAPHRAGM	⊔	56	9-3	346
4a2	STIRRUPS - INT. DIAPHRAGM	⊔	48	7-3	233
4f	PAVING NOTCH	⌐	64	3-7	153
5d4	PAVING SUPPORT - LONGIT.	—	2	31-8	66
	TOTAL REINF. STEEL - LBS.				22452

ESTIMATED SUPERSTRUCTURE QUANTITIES

ITEM	UNIT	TOTAL
STRUCTURAL CONCRETE - CLASS D	CU. YDS.	90.0
REINFORCING STEEL	LBS	22,452
ALUMINUM RAIL	LIN. FT.	169.67
PRETENSIONED PRESTRESSED CONC. BEAMS	NO.	5



BEARING DETAILS



FLOOR DRAIN DETAILS AND LOCATION

SINGLE SPAN 86' × 32' PRETENSIONED PRESTRESSED CONCRETE BRIDGE

ALUMINUM RAIL — CONCRETE ABUTMENTS

SUPERSTRUCTURE DETAILS

STATION 156 + 29 — COUNTY PROJECT LFM-1856(2)--73-42

HARDIN COUNTY

HERBERT A. ARTHUR - CONSULTING ARCHITECT-ENGINEER
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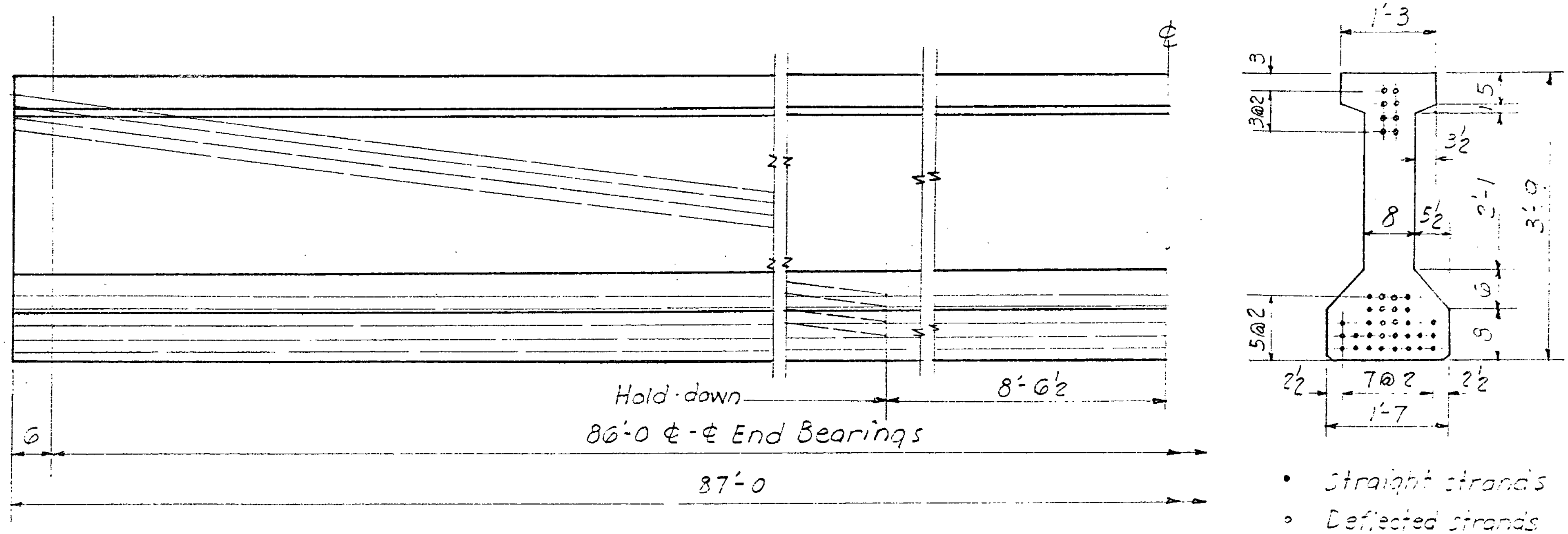
AMES IOWA

NOVEMBER 1967 — SHEET 4 OF 5

DESIGN NO. 168 — HARDIN COUNTY — FILE NO. 23087



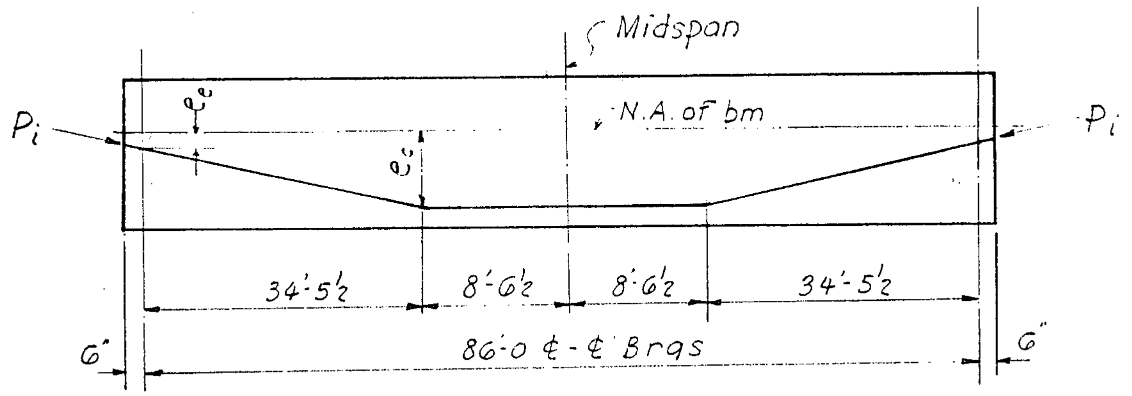
BEAM DATA		
Total initial prestress	Kips	867
Strand size (270^k)		½"φ
No. of straight strands		22
No. of deflected strands		8
Camber ①	in.	2.5 / 3.2
D.L. deflection ②	in.	3.0 / 0.5
Concrete	c.y.	11.62
Beam weight	Tons	18.35

① Upper figure is the beam camber at release. Lower figure is the anticipated beam camber just before slab is placed.

② Upper figure is the elastic deflection of beam due to wt. of slab. Lower figure is the anticipated additional deflection due to creep & shrinkage.

Gross beam area	519.5 □"
Transformed beam I @ ℄ n = 8	120,109 in^4 y_t = 25".47 y_b = 19".53
Transformed beam I @ end n = 8	120,689 in^4 y_t = 25".07 y_b = 19".93

DESIGN CAMBER AND DEFLECTION AT MIDSPAN



(1) Midspan camber at release of prestress:

$$\Delta_1 = \frac{(0.97\ Pi)(L^2)}{(E_{c1})(I_B)} (0.0983e_c + 0.0267e_e)$$

$$- \frac{5}{48} \frac{(M_B)(L^2)}{(E_{c1})(I_B)}$$

Pi = total initial prestressing force in lb.

0.97 Pi = assuming 3% loss of initial prestressing force due to stress relaxation in steel before release.

L = beam span length in inches.

E_{c1} = 2.91×10^6 psi at f'c = 4,500 psi

I_B = average transformed I of beam in inch.^{4}

M_B = moment at midspan (in-lb.) due to wt. of beam.

$$\Delta_1 = \frac{(0.97)(867{,}000)(86 \times 12)^2}{(2.91 \times 10^6)(120{,}400)} (0.0983 \times 14.33$$

$$+ 0.0267 \times 6.2) - \frac{5}{48} \frac{(4{,}803{,}702)(86 \times 12)^2}{(2.91 \times 10^6)(120{,}400)} = 2.5'' \uparrow$$

(2) Midspan camber at time just before placing concrete slab:

$$\Delta_z = \left[\frac{(0.82\ Pi)(L^2)}{E_{c2}\ I_B} (0.0983e_c + 0.0267e_e) \right.$$

$$\left. \frac{5}{48} \frac{(M_B)(L^2)}{(E_{c2})(I_B)} \right] (C)$$

0.82 Pi = assuming 18% loss of initial prestressing force due to stress relaxation in steel, and creep and shrinkage in concrete.

$E_{c2} = 3.06 \times 10^6$ psi at f'c = 5,000 psi.

C = coefficient for creep effect = 1.8

$$\Delta_z = \left[\frac{(0.82)(867{,}000)(86\text{x}12)^2}{(3.06\text{x}10^6)(120{,}400)} (0.0983\text{x}14.33 \right.$$

$$\left. + 0.0267\text{x}6.2) - \frac{5}{48} \frac{(4{,}803{,}702)(86\text{x}12)^2}{(3.06\text{x}10^6)(120{,}400)} \right] 1.8 = 3.2'' \uparrow$$

(3) Midspan deflection due to weight of slab:

$$= \frac{5}{48} \frac{(M_s)(L^2)}{(E_{c2})(I_B)}$$

M_s = moment at midspan (in-lb) due to wt. of slab.

$$= \frac{5}{48} \frac{(6{,}789{,}528)(86\text{x}12)^2}{(3.06\text{x}10^6)(120{,}400)} = 2.04'' \downarrow$$

(4) Midspan deflection due to creep and shrinkage in slab:

$$= 25\% \text{ x } 2.04'' = 0.5'' \downarrow$$