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DEPARTMENT OF TRANSPORTATION ☐ OFFICE OF STRUCTURES DESIGN

Long span bridge deflections

final report

Caltrans

December 1983

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16. Abstract

This report discusses an investigation of the immediate and long term deflection of simple span and continuous prestressed concrete bridges. The study examines the design assumptions and construction methods used to plan and build the superstructure with respect to its deflections. Factors affecting bridge deflection such as concrete strength, modulus of elasticity, aggregate shrinkage, structure shortening, length of time the structure is on falsework, skew, and end condition of the abutments are also analyzed. Falsework load distribution is studied.

Conclusions are as follows: Judgement differences in design assumptions can cause substantial variations in calculated deflections. Maximum falsework loads occur several days after the top deck is cast. Most structures lift off the falsework at midspan when stressed. Most structures deflect elastically and continue to deflect plastically after the falsework is released. Occasionally a structure will rise significantly after a long period of constant deflection.

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This report was prepared in the Special Projects Branch of the Office of Structures Design under the direction of Mr. Guy D. Mancarti and later Mr. W. J. Jurkovich. Contributions of data were made by some 21 resident engineers at various construction sites throughout California. Laboratory work was conducted under the direction of Mr. Lee Wilson. The efforts of all contributors are appreciated.

The data gathering and writing of the first draft of the report were done by Barry H. Nelsen. The later updating and completion of the report were done by Paul J. Jurach.

INTRODUCTION

Accurate prediction of the deflection of a highway bridge is a difficult problem bridge engineers have faced for many years.

Resident engineers on Caltrans construction projects are provided with the dead load deflection in the form of camber diagrams on the contract plans. They superimpose anticipated falsework settlements on this diagram to determine the total required falsework camber. A number of assumptions are made by both designer and resident engineer in determining this total camber.

In past years, Caltrans resident engineers submitted deflection measurements made during the construction of their bridges. This data was then analyzed and evaluated to determine a value for the modulus of elasticity (Ec) for concrete to be used in calculating anticipated deflections. A modulus of elasticity of 3×10^6 psi for reinforced concrete, and 3.5×10^6 psi for prestressed concrete, was then selected. These values were then used to calculate deflections of all concrete bridges. It was found, however, that the actual deflection of many structures was very different from that calculated; consequently, there was a need to determine more accurate value for Ec, to evaluate additional factors that affect deflection and to develop techniques which would more accurately predict concrete structure deflections.

It has been found that at many recently constructed simple span cast-in-place prestressed structures, there has been a significant change in falsework loading during the top slab curing period and well before the stressing operation. The increased loads on some falsework bents have substantially exceeded design values and caused some crushing of falsework columns and caps and have caused some cracks in the soffit slab although these cracks closed up later under prestressing.

Most of the problems that have occurred to date have been on structures requiring falsework openings, where rigid connections and unyielding supports are the rule of the day. In this situation, we might visualize the intermediate supports as springs, yielding under load and transferring additional load to the unyielding supports adjacent to the opening.

There was a need to understand the causes of the changes in falsework loading in order to eliminate the cracking of the soffit slab.

OBJECTIVE

This research project was planned to determine those factors which affect the immediate and long-term deflection of long span (over 100' long) concrete bridges. After the project started, it was decided to also investigate the apparent problem of significant changes in falsework loads during the concrete curing period.

Deflections

It was planned to investigate the effect of the following factors on deflections:

1. Concrete strength.
2. Elastic modulus (Ec).
3. Aggregate type.
4. Structure shortening.
5. Length of time before striking falsework.
6. Skew.
7. End condition of abutments.
8. Creep.
9. Stiffness.

The project was also expected to develop charts or curves which would correlate the effect of increasing skew on deflection.

The findings, conclusions and recommendations of the research were to be presented to the Office of Structures Design for their consideration and implementation.

After implementation it was expected that the research results would provide:

1. An improvement in deck riding quality.
2. Increased safety.
3. Improved aesthetics.
4. Improved deck drainage.

Falsework Loading Changes

The project was expected to investigate the degree of changes in falsework loading that regularly occur with normal longitudinal shrinkage of the top slab and to recommend procedures to eliminate the kind of damage to falsework and structures which had been occurring.

RESEARCH PROCEDURE

Deflections

Just prior to this project, it was determined that most future long span structures would be:

1. Prestressed cast-in-place (CIP) simple spans.
2. Prestressed CIP continuous spans.
3. Reinforced concrete continuous spans.

Some 25 structures which were soon to be constructed were selected for testing. These bridges had a total of 32 spans that ranged from 118.3' to 200.0' in length. These structures are listed in the appendix on pages A1-A3.

Data collection forms and instructions were prepared and distributed to those projects with the 25 selected bridges.

Field test work included preparation of 8 concrete cylinder samples during each soffit and stem pour and 8 more during the top deck placement. These cylinders were prepared as specified by Test Method Calif. No. 539-A. Field curing was done as specified in Method 2 of the Test Method Calif. No. 540-B (see Appendix). They were sent to the Caltrans Laboratory as soon as allowable and then cured in the fog room until tested. Strength and modulus of elasticity were determined as specified by ASTM Test Methods C 192 and C 469-65 (reapproved 1970). Two cylinders each were broken at 7 days, 14 days, 28 days, and 60 days.

During construction, reference nails were placed at the abutment, 1/4 span, 1/2 span and 3/4 span of each structure, at approximately 6 inches inside each railing. Elevations of, and distances between, these nails were recorded as follows:

1. Before prestressing.
2. After prestressing.
3. After falsework release.
4. One week after falsework release.
5. One month after falsework release.
6. Three months after falsework release.
7. Six months after falsework release.
8. One year after falsework release.
9. Two years after falsework release.
10. Four years after falsework release. (Elevations only.)

Reference points were also placed on abutment footings. Distances were measured from these points to the front face of the abutment diaphragm at the above specified times until backfill operations covered the points. The purpose of these measurements was to see how much the abutment diaphragm slides on top of the abutment footing. This movement was compared to the superstructure shortening after prestressing. Differences in horizontal movement between the superstructure and the abutment footing indicated positive or negative moments in the abutment diaphragm.

The final deflection analysis was made on data from some 20 representative structures with a total of 26 spans (18 single span structures and 2@ 3 span structures). Deflection data was taken from both the left and right curblines making some 52 sets of deflection data to be evaluated.

Falsework Loading Changes

Twelve Enerpac LH-5006 100,000-pound (45,359Kg) capacity mini-hydraulic load cells were purchased to measure loads on falsework posts during construction. See Figure 23.

Three structures were selected for testing and selected bents were instrumented as the falsework was constructed on the bridges as follows:

1. Allendale Road Undercrossing, Bridge No. 23-169R

   This bridge is a 90' (27.43m) simple span cast-in-place prestressed concrete box girder.

   The falsework system consisted of steel stringers supported on timber posts and bent caps, Figures 24, 25 and 26. Falsework bents 2 and 5 were founded on compacted fill material. Falsework bents 3 and 4 were founded on compacted aggregate base material. The contractor removed several feet of existing roadway surface at the bridge site. About 1' (.3m) of aggregate base material was then placed as a temporary roadway surface and foundation for the falsework bents adjacent to traffic.

   The workmanship used in falsework erection was rated excellent.

   Load cells were placed in falsework bents 3 and 4 as shown in Figures 26 and 27. A 12" x 12" x 3/8" (.3m x .3m x 9.5mm) steel distribution plate was placed on the top and bottom of each load cell. These plates prevented local overstressing of the timber falsework post bearing area.

   Falsework post settlements were determined using a Zeiss level and sections of carpenter's rule nailed to the posts. Load cell readings were made in the mornings at regular intervals. Additional readings were made in the afternoons after it was noticed that readings varied throughout the day.

   Bridge construction chronology was as follows:

Cast girder stems and bottom slab	March 16, 1973
Cast top deck	April 6, 1973
Prestress bridge	April 16, 1973
Release falsework	April 19, 1973

2. M Street Undercrossing, Bridge No. 42-228R

This structure is a 130' (39.62m) simple span cast-in-place prestressed concrete box girder.

The falsework system is shown in Figures 31 and 32. The workmanship used in falsework erection was rated satisfactory.

Load cells with steel distribution plates were placed in bents 2 and 3, Figure 32. Load cell readings and falsework post elevations were taken at regular intervals during the pour operation and on a daily basis until the falsework was removed.

Load cell readings taken following the stem pour indicated substantial increases (25-30%) between the morning and afternoon readings. (Figure 33)

To determine the effect of temperature changes, 25 pounds (11.3Kg) of ice was applied to each load cell. Lower readings were obtained after icing the cells. Readings were taken in the mornings and afternoons without ice at all times to maintain consistency. Load cell readings are recorded in Tables A19 through A31.

Bridge construction chronology was as follows:

Cast girder stems and bottom slab	May 31, 1973
Cast top deck slab	July 20, 1973
Prestress bridge	August 7, 1973
Release falsework	August 13, 1973

3. Old Oregon Trail Undercrossing, Bridge No. 6-178L

Falsework for this 146' (44.5m) simple span cast-in-place prestressed concrete box girder was built on compacted base material. The falsework was a combination of timber posts and caps with steel wide flange stringers, Figure 34 and 35. Erection workmanship was rated average.

Load cells and bearing plates were placed under falsework posts in bents 3, 4, and 5 as shown in Figure 36. Load cell and settlement readings were recorded from bottom slab and stem pour until after the structure was prestressed.

Bridge construction chronology was as follows:

Cast bottom slab and girder stems	August 30, 1974
Cast top deck slab	September 17, 1974
Prestress bridge	September 30, 1974
Release falsework	October 5, 1974

TESTING RESULTS

Deflections

Elevations of the reference nails and distances between them were taken by field engineering crews as noted under Research Procedures. These measurements were taken with available field equipment such as Zeiss levels and Linker rods and using average care. Rod readings were estimated to 0.001'. Distances between points were measured by applying 25 lbs. tension to a 200' steel chain lying on the bridge deck. These measurements were estimated to 0.001'. Elevations of the structure were first taken just before the structure was stressed. These elevations were used as reference points to compare future readings.

Varying work requirements on the different projects determined the exact time of elevation readings. These differences in work load occasionally caused significant variations in the time between deflection readings. The contractors methods and procedures of removing falsework also affected the scheduling of deflection readings.

Elevations at the right and left edge of deck at midspan of each structure were plotted on 4-cycle semi-log paper (deflection versus time). These elevations were adjusted to compensate for the settlement of the abutments and bents. Settlements of up to 0.02' (4 years) at the abutments were common. The maximum settlement of an abutment was .14' (4 years). Settlements of the bents in the only continuous three span bridge (120'-200'-120') were .04' (4 years).

Figures A46 thru A85 show deflection data gathered to date. In most cases the bridge lifted off the falsework at midspan upon stressing. Settlement charts indicated that uplift is usually fully relieved when the falsework is released, however a few bridges remained above the "before stressing" position after the falsework was released. Figures 1 and 2 summarize data from some typical bridges. A detailed discussion of the "deflection at centerline of span" curves is made on Pages 56-61.

Conversations with field engineers have revealed a common reluctance to build negative camber into a bridge. They hesitate to build in a sag for fear it will not "come out." Plots of elevations on Market Street Off-Ramp OC. Br. #57-842 (see Figures A80-A84) indicate these fears, in this case at least, are not justified as the entire .10' of negative camber has "come out" in 4 years. The sag spans in this structure are the end spans of a 3 span continuous structure in which the center span had a positive camber of .20' and the full .20' camber has also "come out" in 4 years.

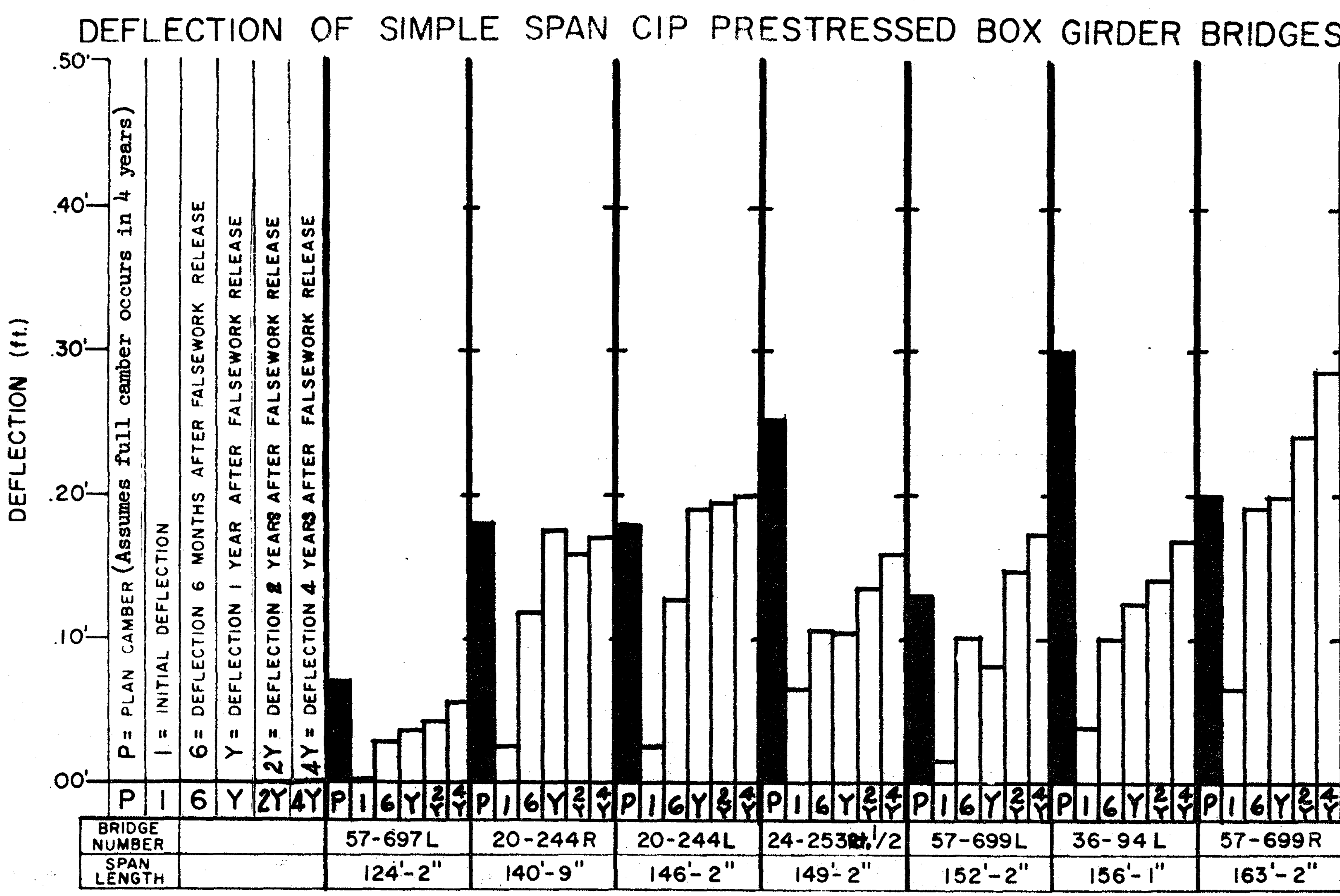

DEFLECTION OF SIMPLE SPAN CIP PRESTRESSED BOX GIRDER BRIDGES
DEFLECTION (ft.)
.50'
.40'
.30'
.20'
.10'
.00'
P = PLAN CAMBER (Assumes full camber occurs in 4 years)
I = INITIAL DEFLECTION
6 = DEFLECTION 6 MONTHS AFTER FALSEWORK RELEASE
Y = DEFLECTION 1 YEAR AFTER FALSEWORK RELEASE
2Y = DEFLECTION 2 YEARS AFTER FALSEWORK RELEASE
4Y = DEFLECTION 4 YEARS AFTER FALSEWORK RELEASE
P I 6 Y 2Y 4Y
BRIDGE NUMBER
SPAN LENGTH
57-697L
124'-2"
20-244R
140'-9"
20-244L
146'-2"
24-253Rt.1/2
149'-2"
57-699L
152'-2"
36-94 L
156'-1"
57-699R
163'-2"

Figure 1

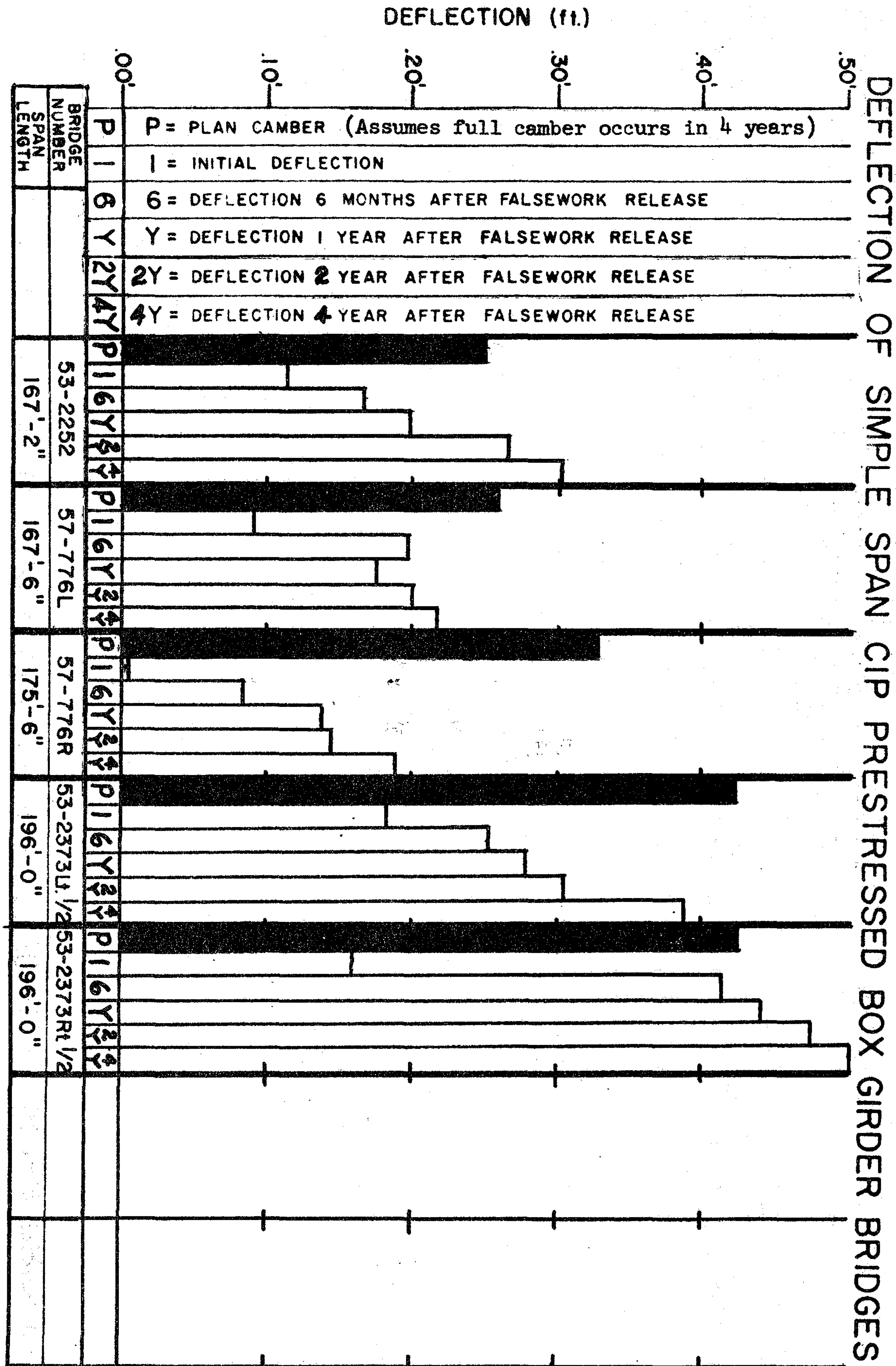

DEFLECTION OF SIMPLE SPAN CIP PRESTRESSED BOX GIRDER BRIDGES
DEFLECTION (ft.)
.00'
.10'
.20'
.30'
.40'
.50'
P = PLAN CAMBER (Assumes full camber occurs in 4 years)
I = INITIAL DEFLECTION
6 = DEFLECTION 6 MONTHS AFTER FALSEWORK RELEASE
Y = DEFLECTION 1 YEAR AFTER FALSEWORK RELEASE
2Y = DEFLECTION 2 YEAR AFTER FALSEWORK RELEASE
4Y = DEFLECTION 4 YEAR AFTER FALSEWORK RELEASE
BRIDGE NUMBER
SPAN LENGTH
53-2252
167'-2"
57-776L
167'-6"
57-776R
175'-6"
53-2373Lt 1/2
196'-0"
53-2373Rt 1/2
196'-0"

Figure 2

Elastic Modulus and Concrete Strength

Concrete cylinders were made and cured in conformance with California Test Method No. 540-B (see page A42-A43). Handling and storage of cylinders conformed to Method 2.

Chord Modulus of Elasticity was computed from test data using ASTM 469.

Modulus of elasticity and strength test results of the various concrete samples were submitted for electronic data processing (EDP). A statistical comparison was made of concrete strength versus age and modulus of elasticity versus age. Scatter diagrams of the accumulated data are shown in Figures 3 thru 16.

Plots of strength versus age show that the 8.5 sack/cy concrete was generally 7% stronger than the 6.0 sack/cy concrete at any age.

Plots of Ec versus age show a difference of generally 25% between the maximum ordinate curve and minimum ordinate curve at any age. Oddly enough, the curves on Figure 16 (which were plotted independently) have the 7.0 sack concrete as the upper limit and the 6.5 sack concrete as the lower limit. At 60 days, the maximum individual Ec value was 6.42×10^6 psi (7.0 sks/cy) and the minimum Ec value was 3.16×10^6 psi (7.5 sks/cy). This indicates the great spread in Ec test values for concrete of similar cement content and the danger of establishing a design Ec from just a few tests.

Aggregate Shrinkage

Shrinkage values and aggregate petrology for most of the aggregate sources used by the State have been determined by the California Transportation Laboratory. These tests were done in accordance with ASTM Procedures.

A partial list of structures and aggregate shrinkage values is shown in the appendix.

Shrinkage values significantly under 0.048% are considered low. Shrinkage values significantly over 0.048% are considered high.

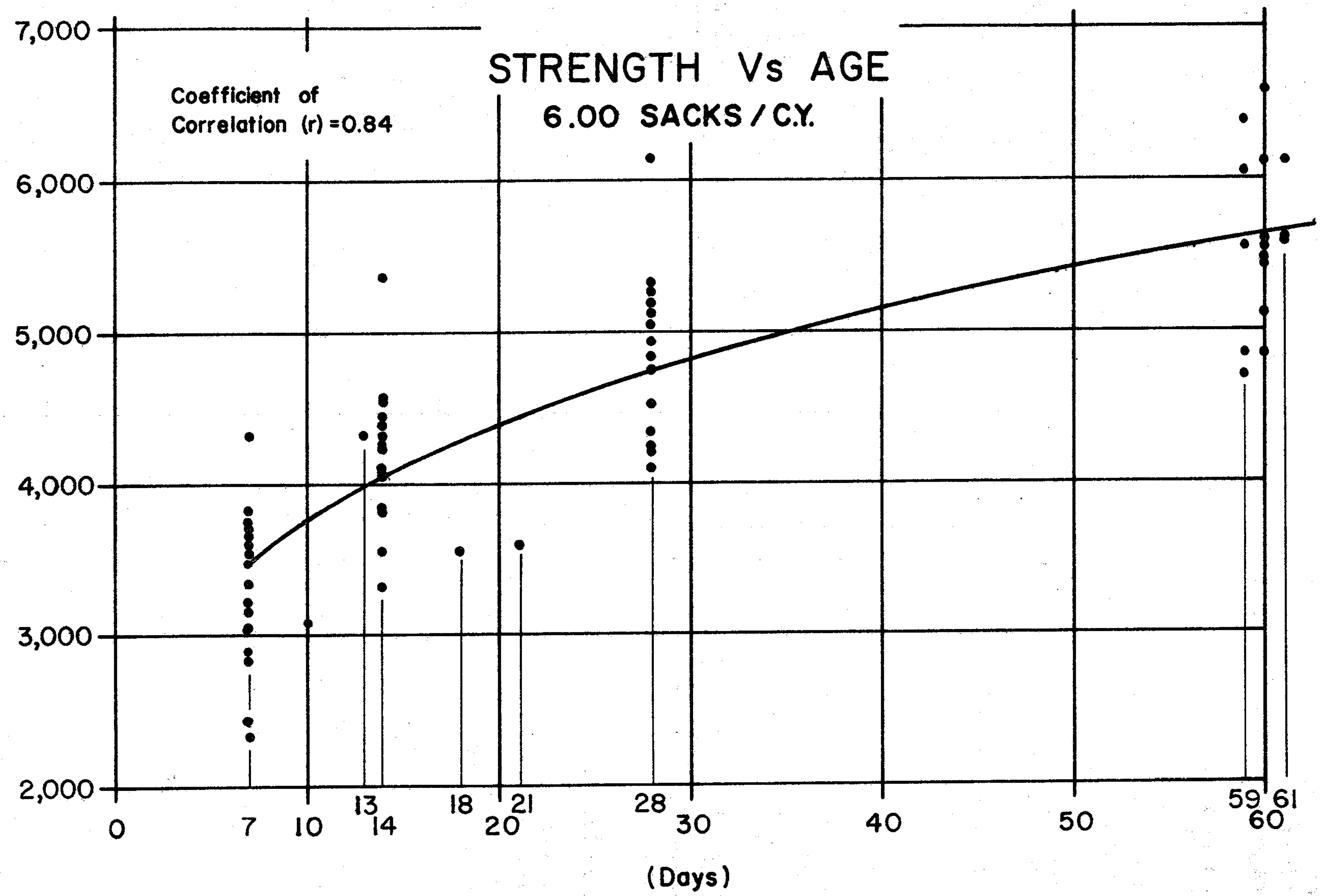

STRENGTH Vs AGE
6.00 SACKS / C.Y.
Coefficient of
Correlation (r) = 0.84
psi
7,000
6,000
5,000
4,000
3,000
2,000
0
7
10
13
14
18
20
21
28
30
40
50
59
60
61
(Days)

Figure 3

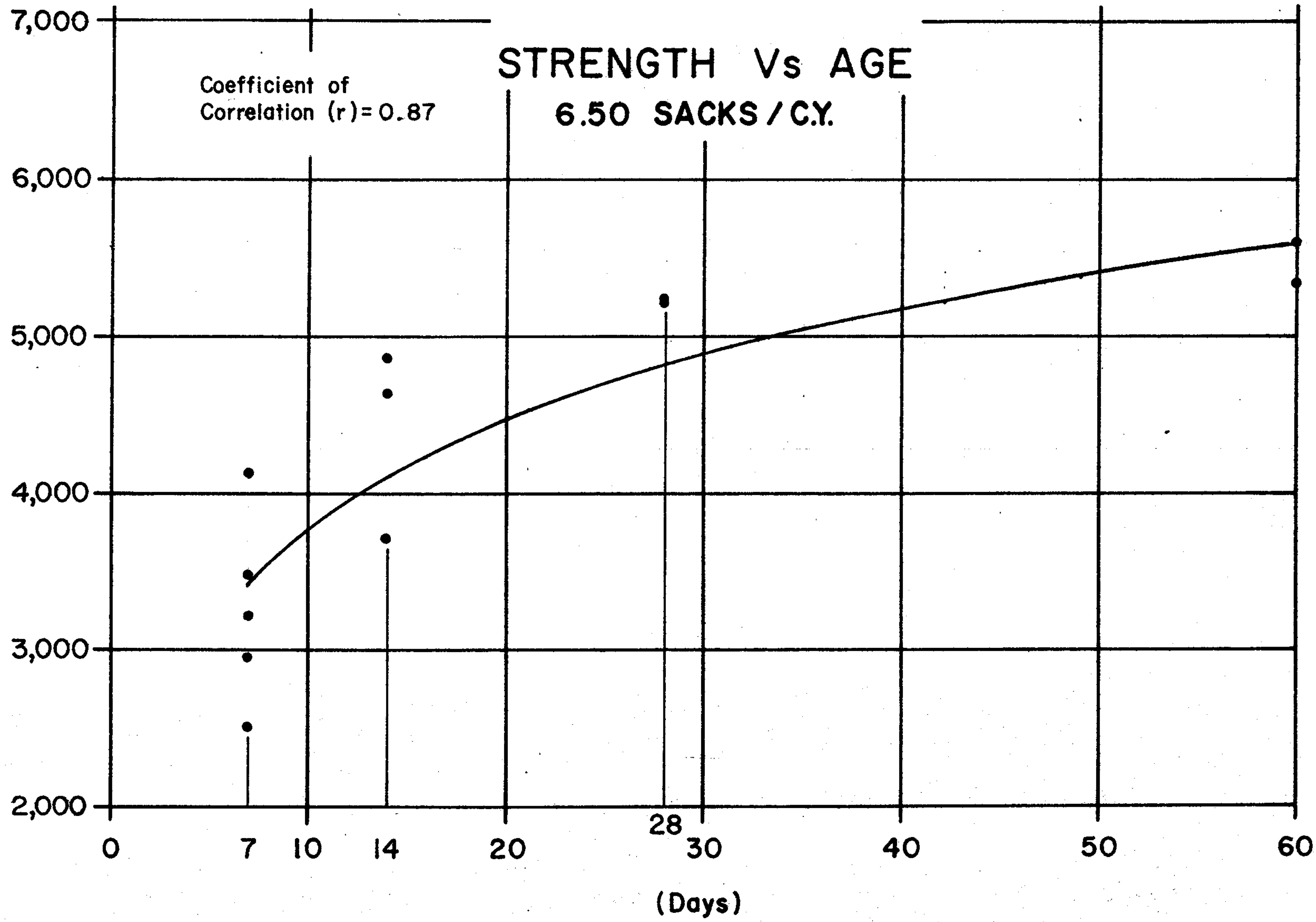

STRENGTH Vs AGE
6.50 SACKS / C.Y.
Coefficient of
Correlation (r) = 0.87
psi
7,000
6,000
5,000
4,000
3,000
2,000
0
7
10
14
20
28
30
40
50
60
(Days)

Figure 4

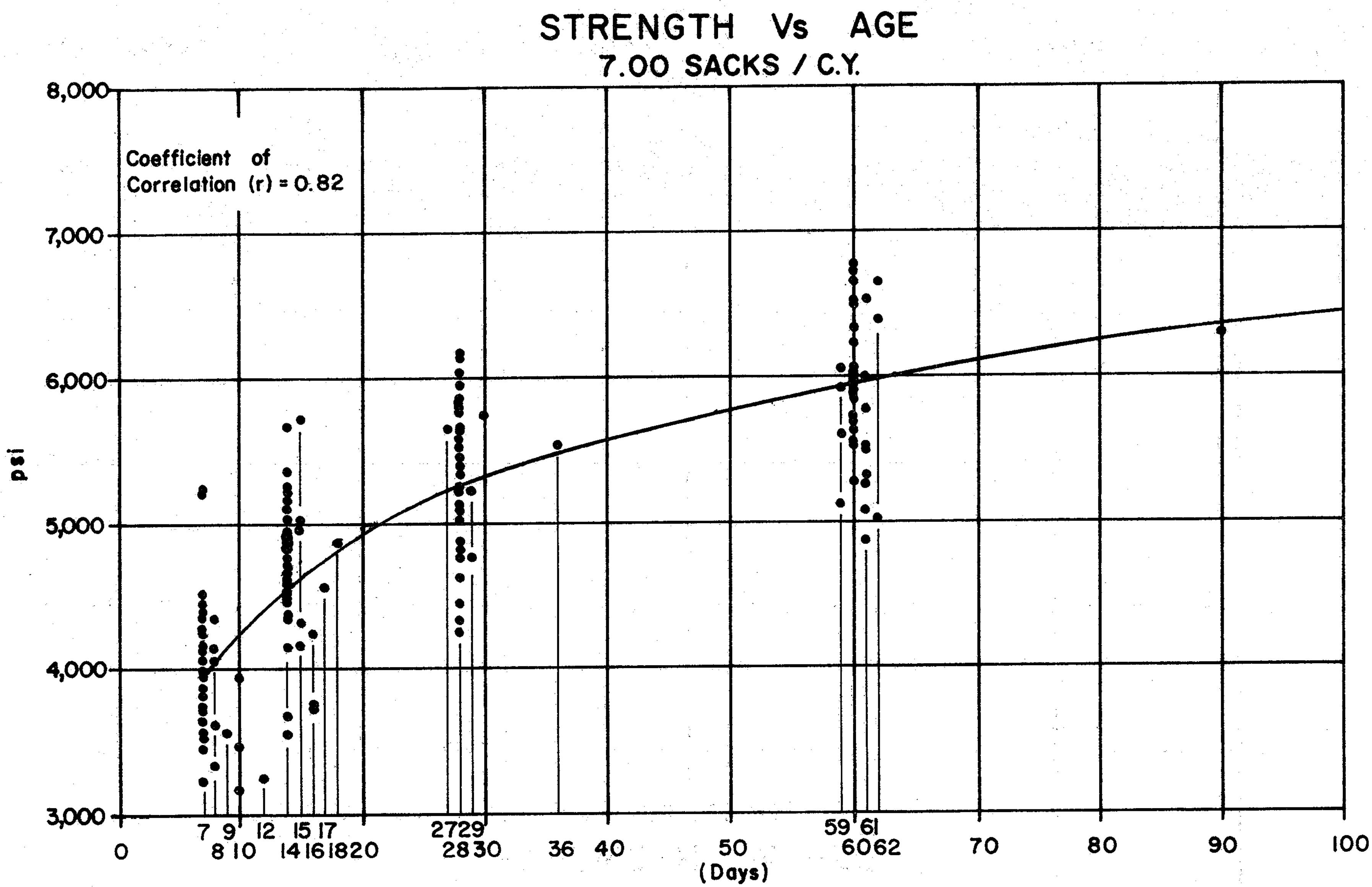

STRENGTH Vs AGE
7.00 SACKS / C.Y.
Coefficient of Correlation (r) = 0.82
psi
8,000
7,000
6,000
5,000
4,000
3,000
0
7 9 12 15 17
8 10 14 16 18 20
27 29
28 30
36 40
50
59 61
60 62
70
80
90
100
(Days)

Figure 5

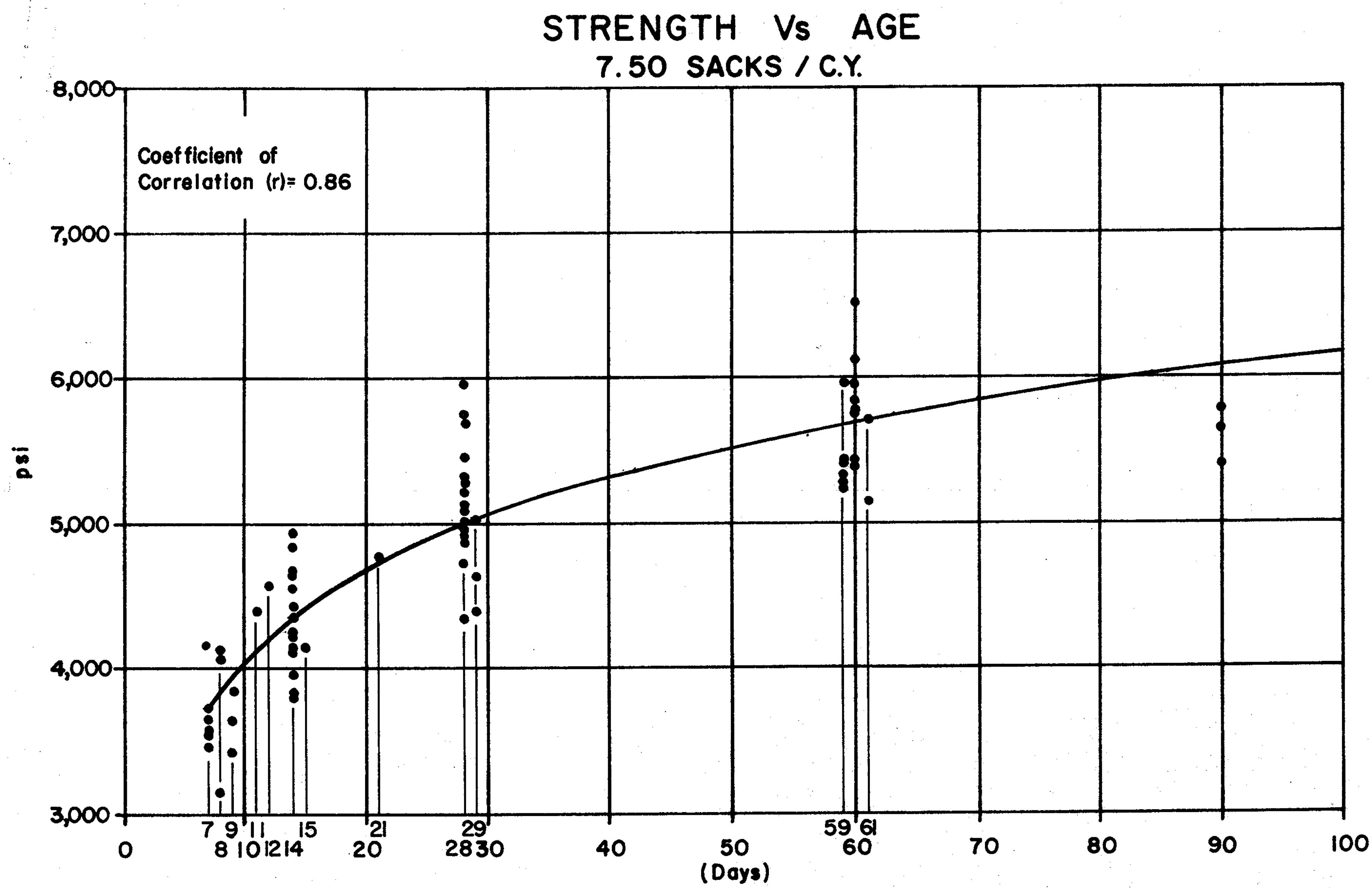

STRENGTH Vs AGE
7.50 SACKS / C.Y.
Coefficient of Correlation (r)= 0.86
psi
8,000
7,000
6,000
5,000
4,000
3,000
0
7 9 11 15 21 29 59 61
8 10 12 14 20 28 30
40
50
60
70
80
90
100
(Days)

Figure 6

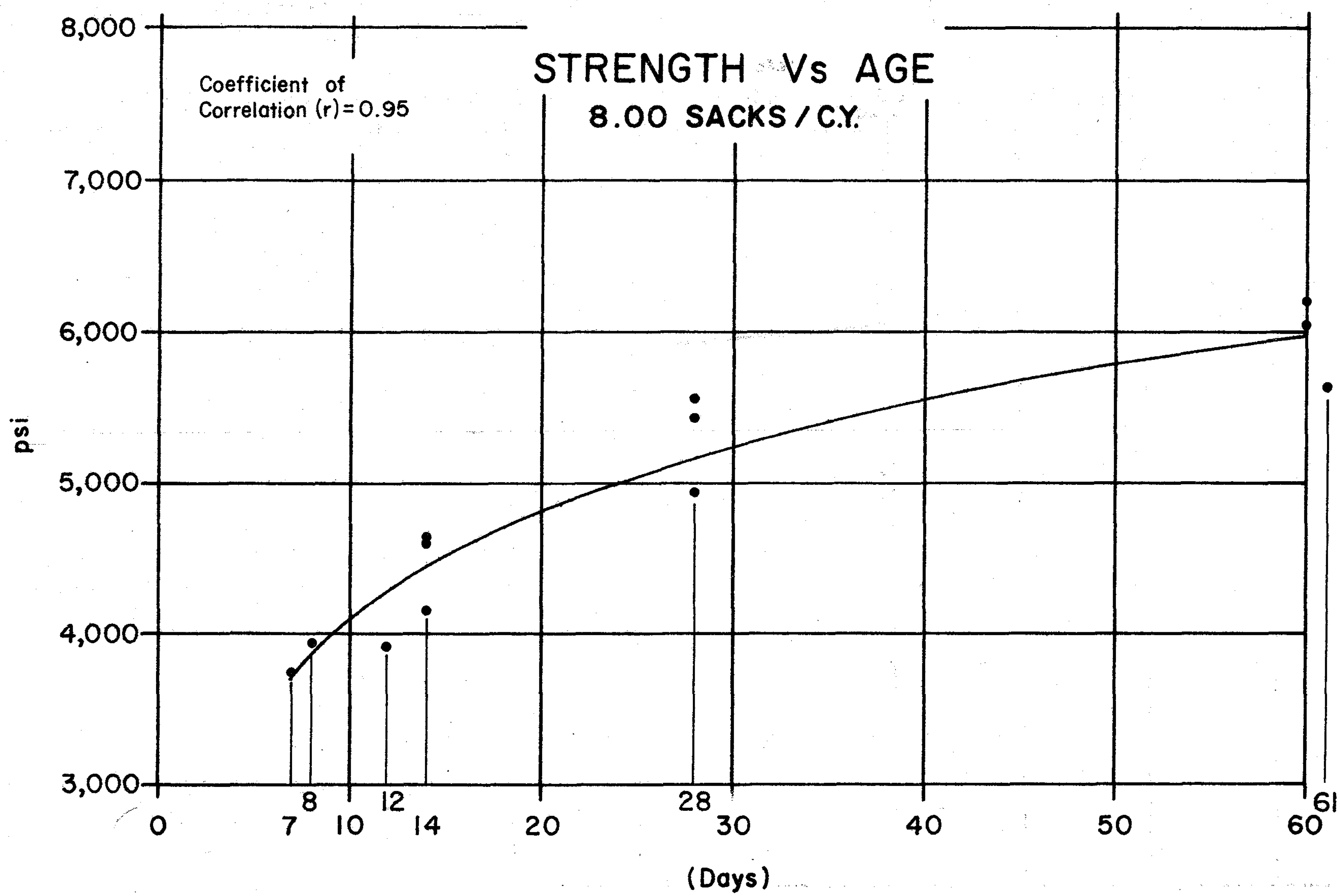

STRENGTH Vs AGE
8.00 SACKS / C.Y.
Coefficient of Correlation (r) = 0.95
psi
8,000
7,000
6,000
5,000
4,000
3,000
0
7
8
10
12
14
20
28
30
40
50
60
61
(Days)

Figure 7

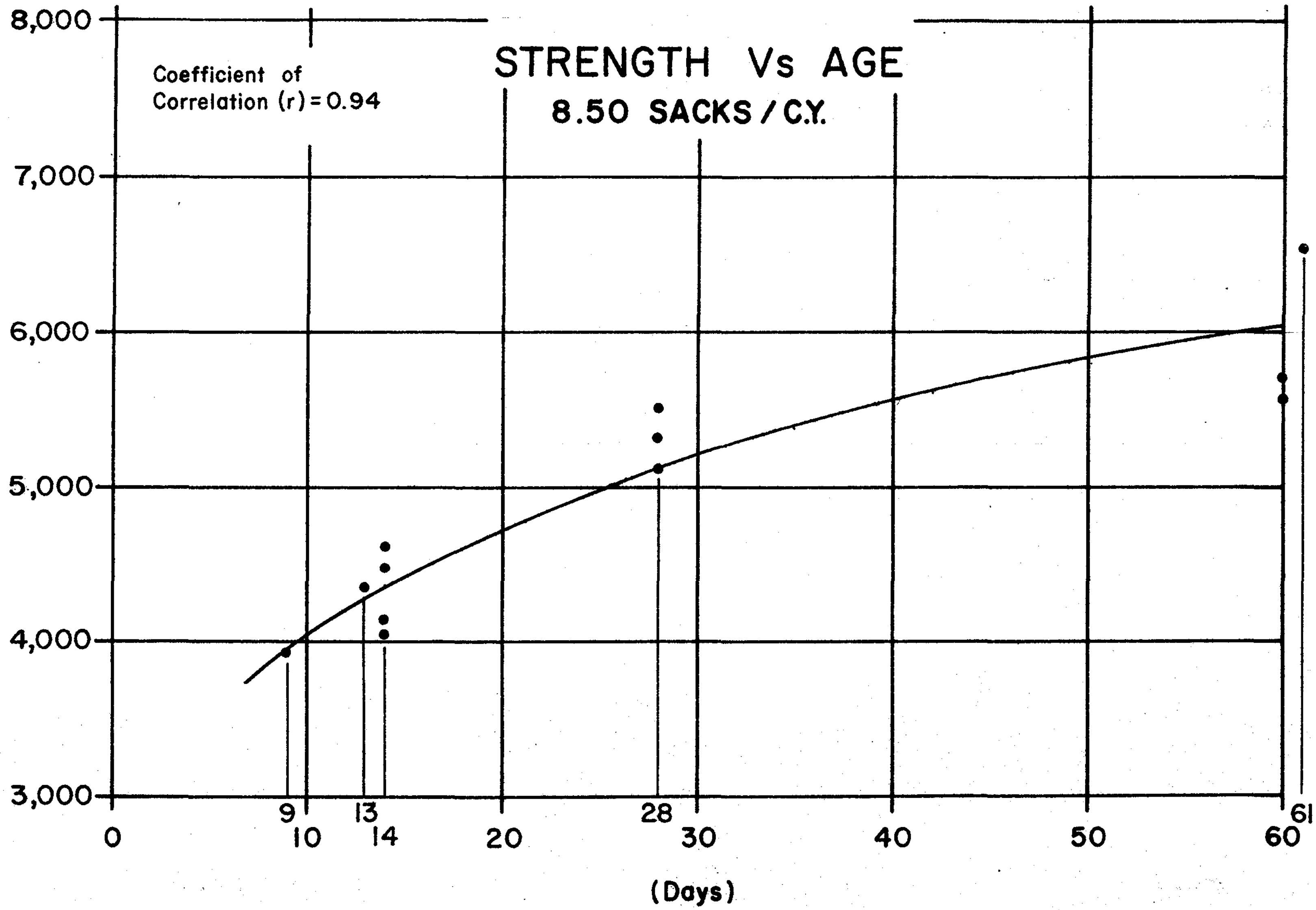

STRENGTH Vs AGE
8.50 SACKS / C.Y.
Coefficient of
Correlation (r) = 0.94
8,000
7,000
6,000
5,000
4,000
3,000
psi
0
9
10
13
14
20
28
30
40
50
60
61
(Days)

Figure 8

SUMMARY
STRENGTH Vs AGE

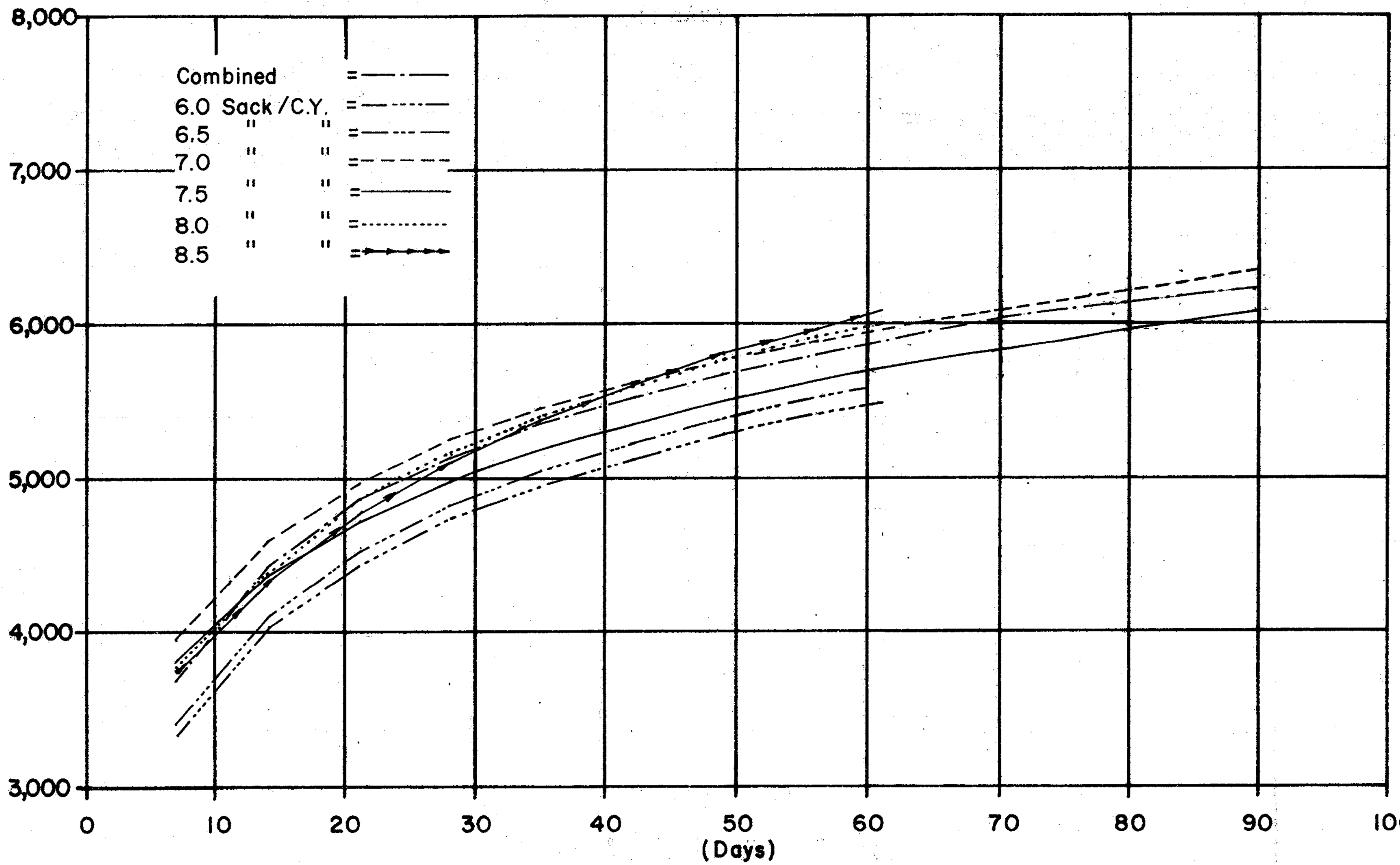
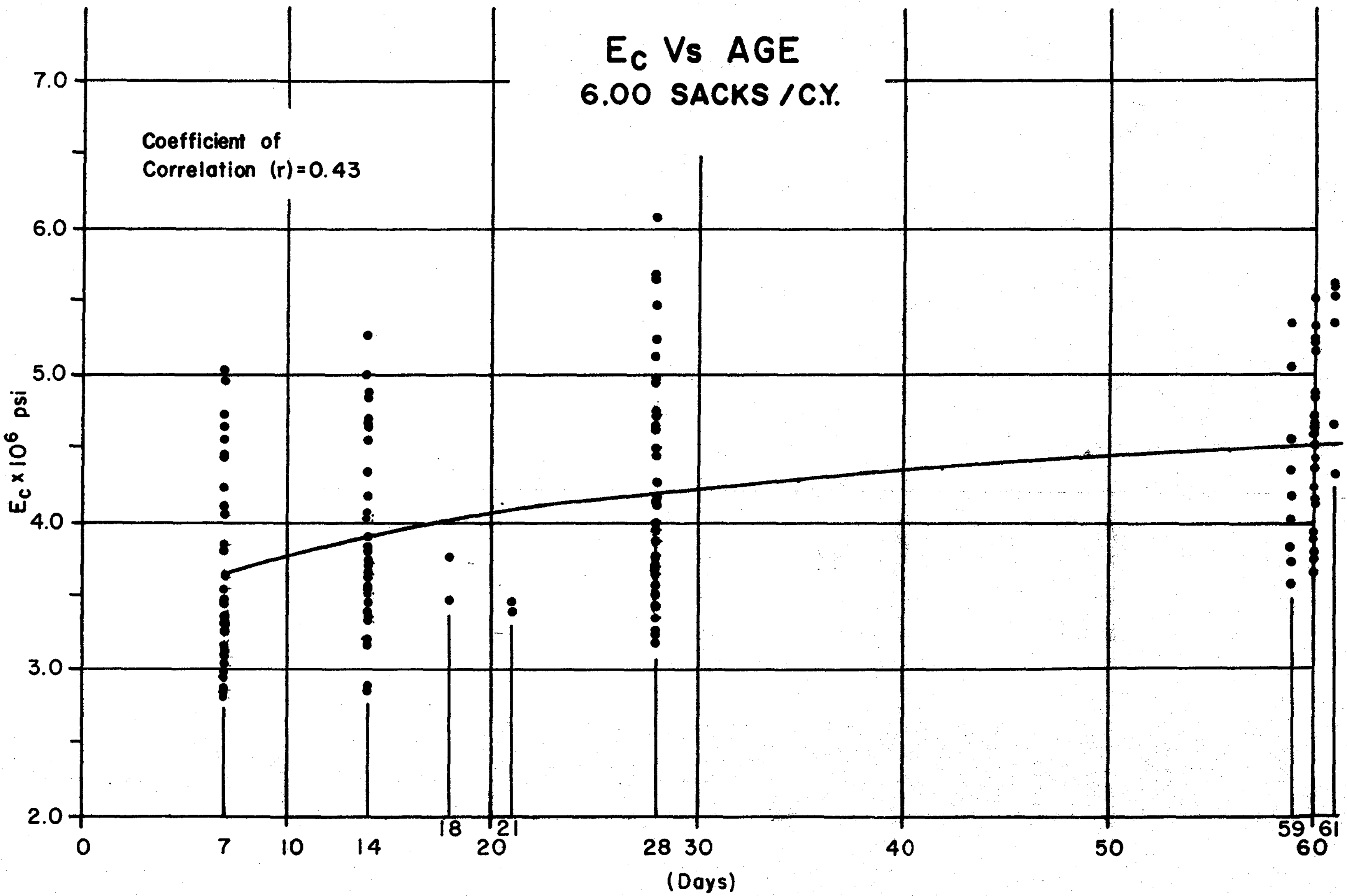
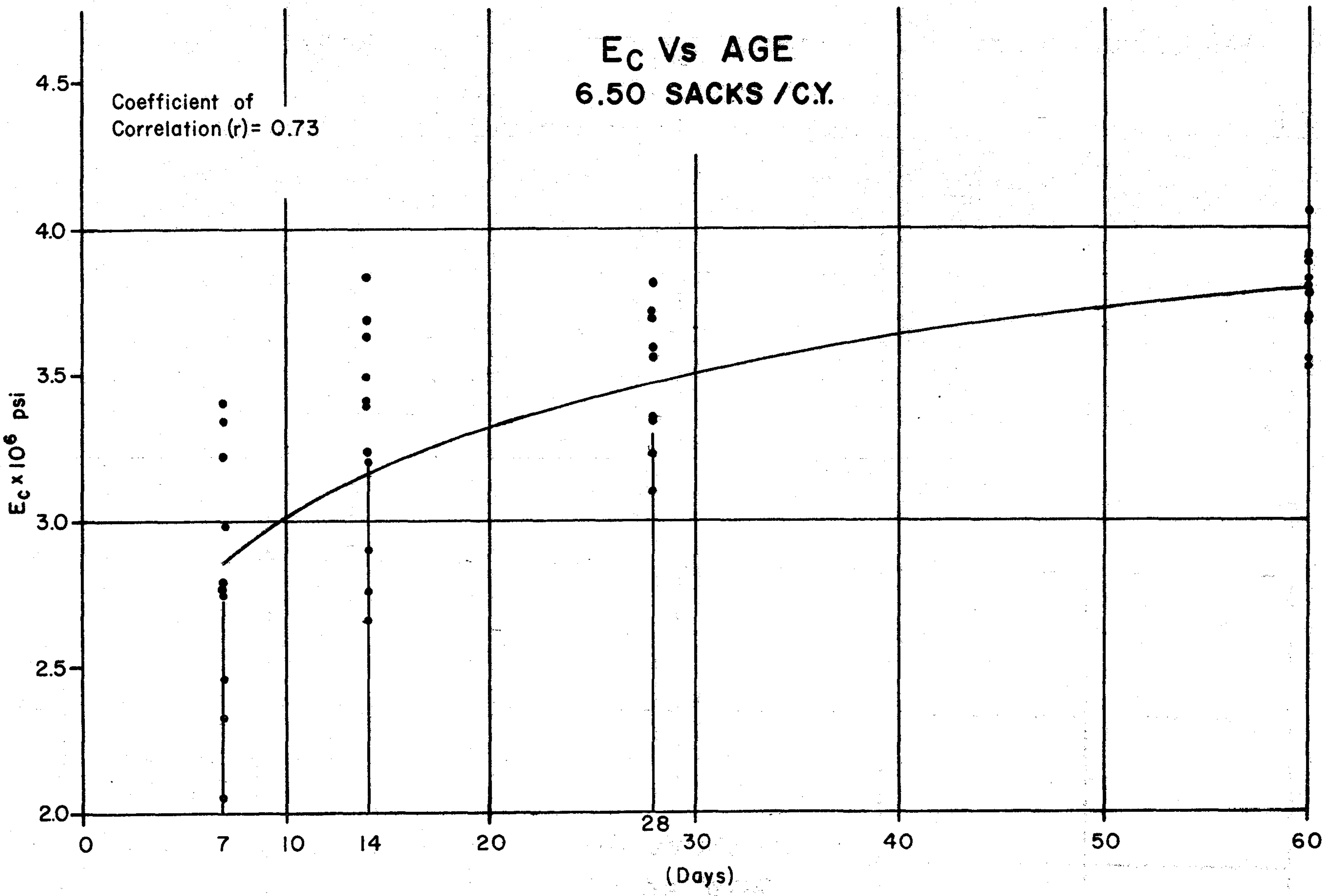


Figure 9



E_C Vs AGE
6.00 SACKS / C.Y.
Coefficient of
Correlation (r) = 0.43
$E_c \times 10^6$ psi
7.0
6.0
5.0
4.0
3.0
2.0
0
7
10
14
18
20
21
28 30
40
50
59
60
61
(Days)

Figure 10



Ec Vs AGE
6.50 SACKS / C.Y.
Coefficient of
Correlation (r) = 0.73
Ec x 10^6 psi
4.5
4.0
3.5
3.0
2.5
2.0
0
7
10
14
20
28
30
40
50
60
(Days)

Figure 11

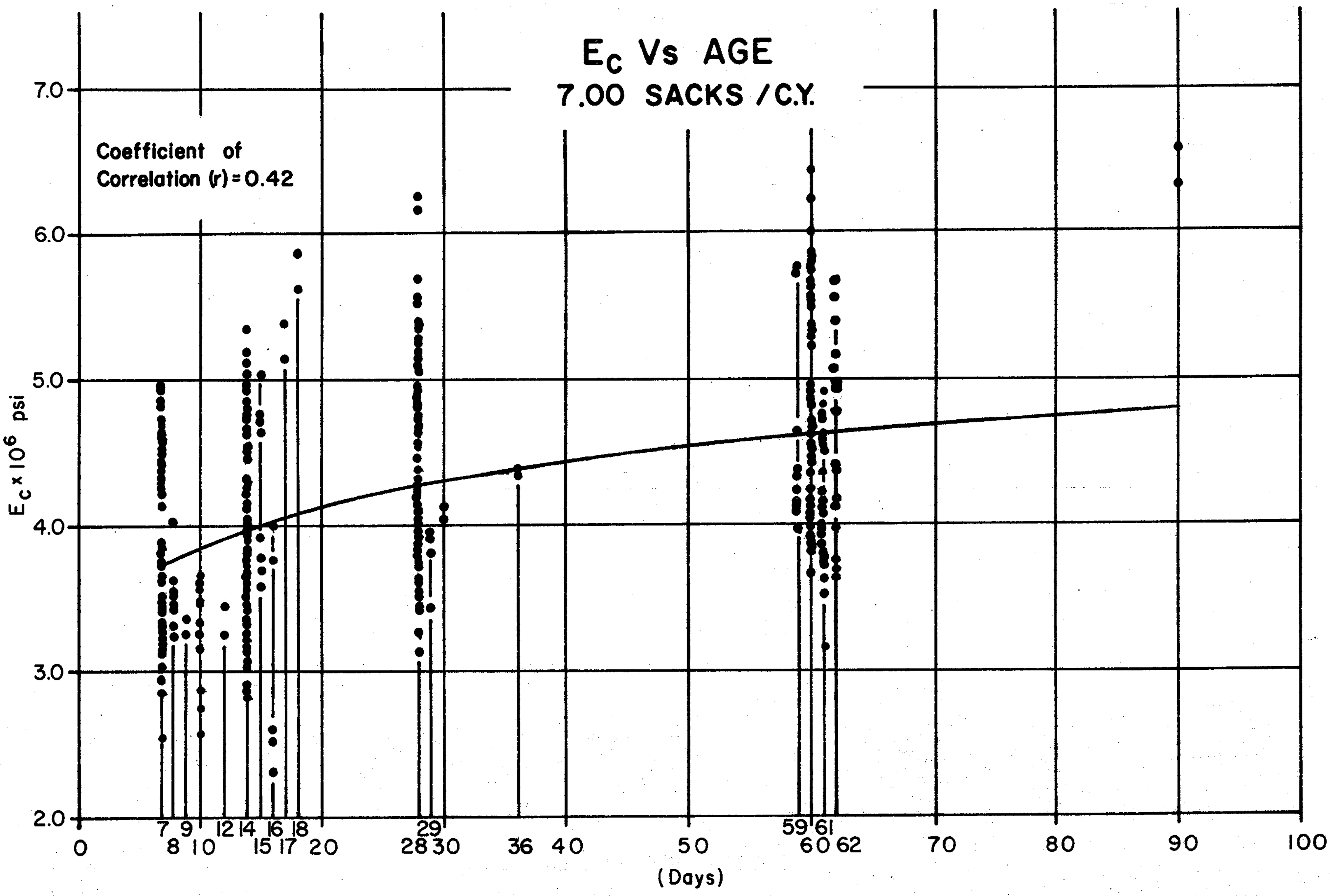
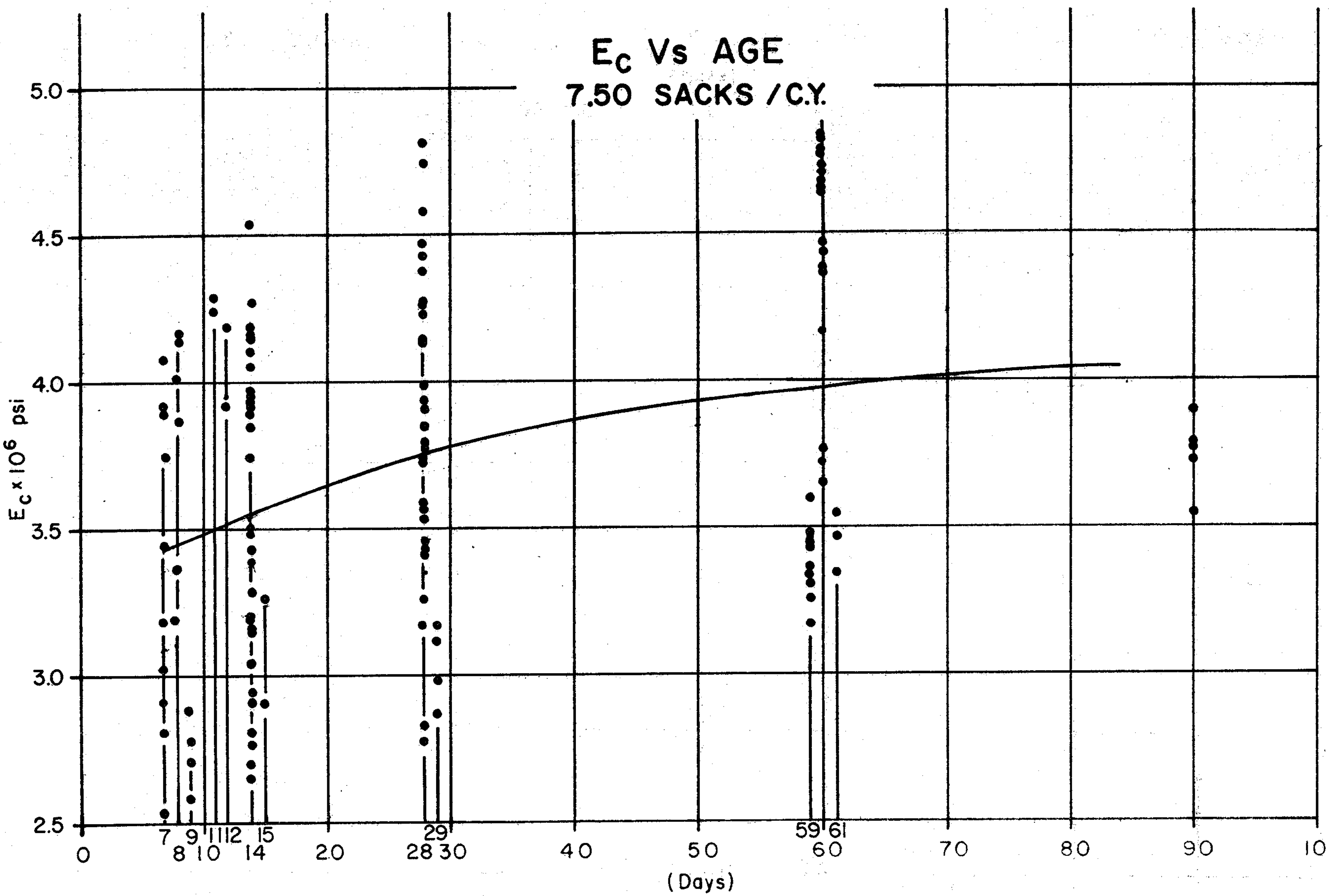

Ec Vs AGE
7.00 SACKS / C.Y.
Coefficient of
Correlation (r) = 0.42
Ec x 10^6 psi
7.0
6.0
5.0
4.0
3.0
2.0
0
7 9 12 14 16 18
8 10 15 17 20
29
28 30
36 40
50
59 61
60 62
70
80
90
100
(Days)

Figure 12



Ec Vs AGE
7.50 SACKS / C.Y.
Ec x 10^6 psi
5.0
4.5
4.0
3.5
3.0
2.5
0
7
8
9
10
11
12
14
15
20
28
29
30
40
50
59
60
61
70
80
90
100
(Days)

Figure 13

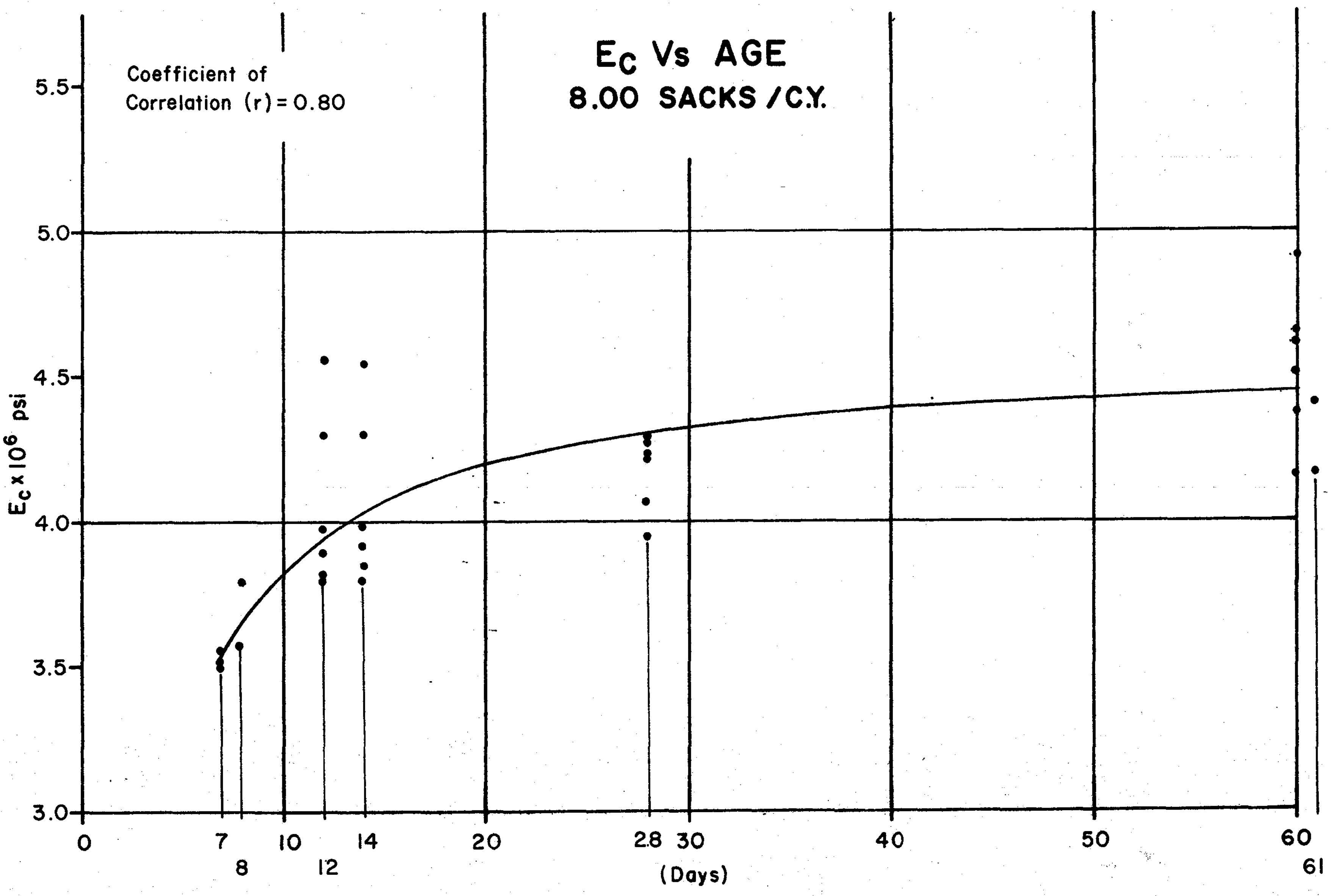
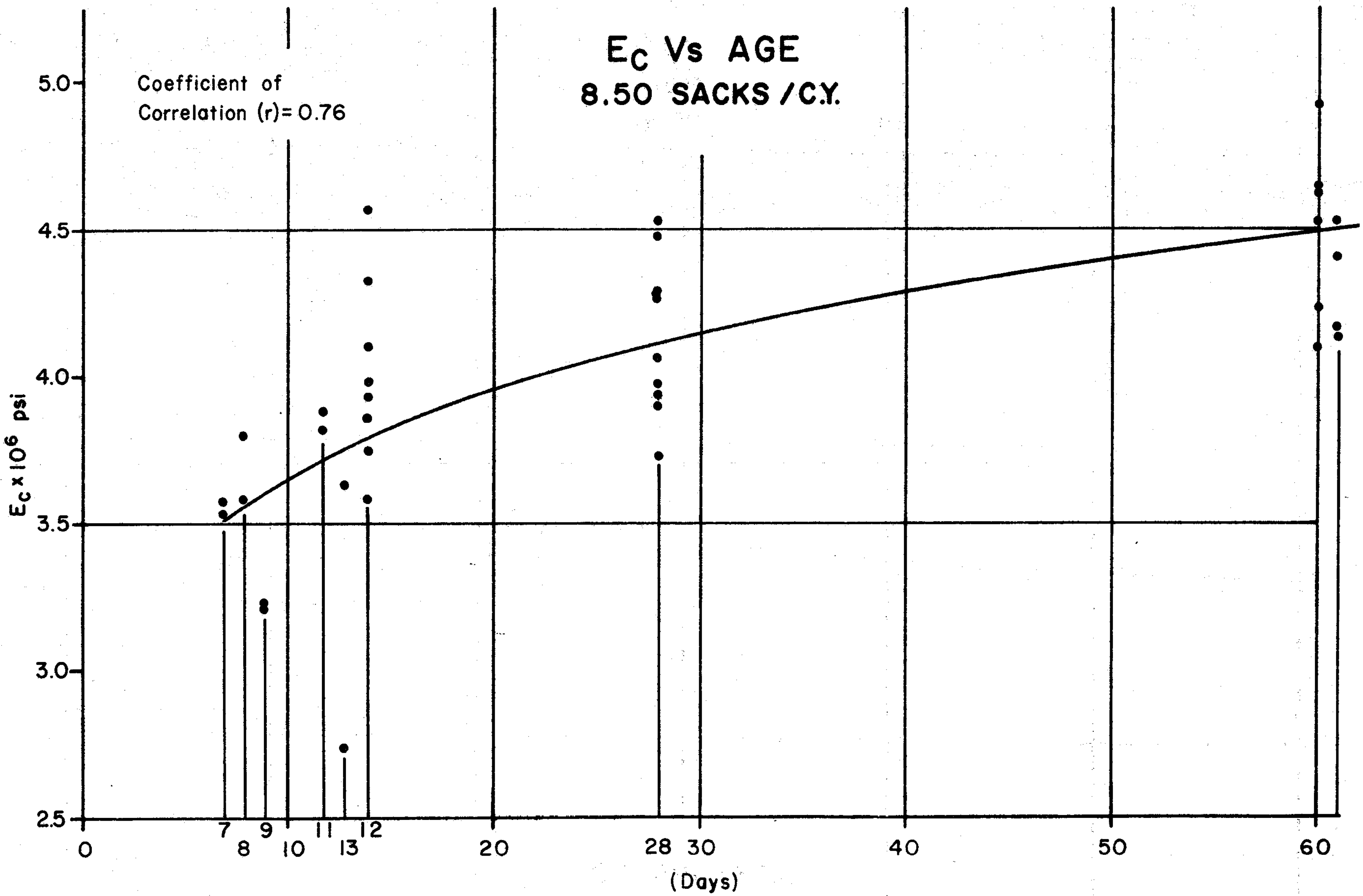

E$_C$ Vs AGE
8.00 SACKS / C.Y.
Coefficient of
Correlation (r) = 0.80
E$_c$ x 10^6 psi
5.5
5.0
4.5
4.0
3.5
3.0
0
7
8
10
12
14
20
28
30
40
50
60
61
(Days)

Figure 14



EC Vs AGE
8.50 SACKS / C.Y.
Coefficient of
Correlation (r) = 0.76
Ec x 10^6 psi
5.0
4.5
4.0
3.5
3.0
2.5
0
7
8
9
10
11
13
12
20
28 30
40
50
60
(Days)

Figure 15

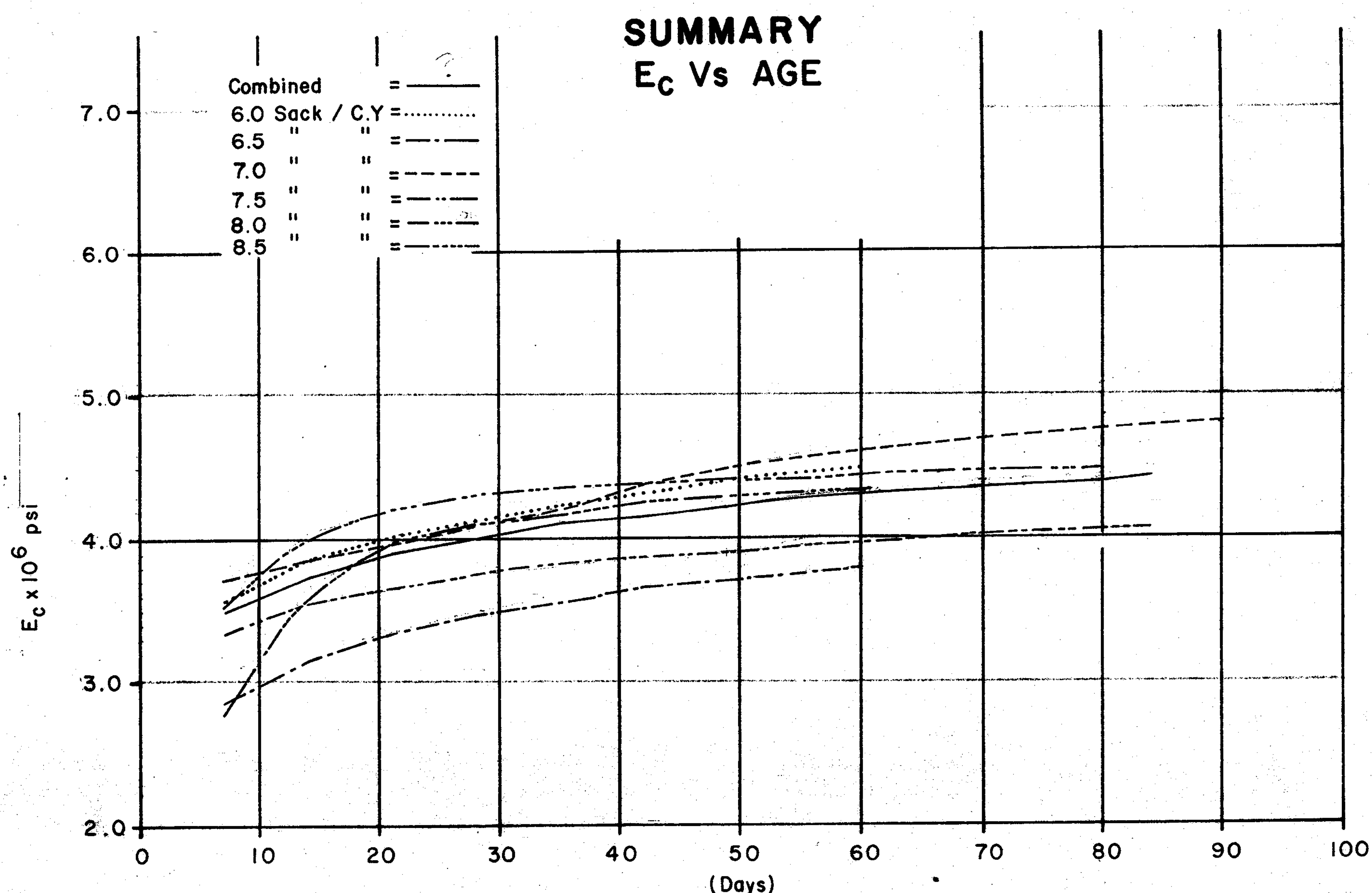

SUMMARY
E_C Vs AGE
Combined =
6.0 Sack / C.Y =
6.5 " " =
7.0 " " =
7.5 " " =
8.0 " " =
8.5 " " =
E_c x 10^6 psi
2.0
3.0
4.0
5.0
6.0
7.0
0
10
20
30
40
50
60
70
80
90
100
(Days)

Figure 16

Structure Shortening

Measurements of deck shortening and abutment movement were not very successful. Clear access to reference points was frequently prevented by curing rugs, deck finishing machines, prestressing equipment, or construction equipment.

Long-term sliding of the abutment diaphragm on the abutment footing was successfully observed on only a few structures. The abutment footing points were usually covered by backfilling operations shortly after the falsework was removed.

Tables 4 thru 10 (pages A4-A10) list measured horizontal movements on several bridges. Measurements on the footings show how much the abutment diaphragm had slid on top of the footing. Most footing measurements indicate that the abutment diaphragm doesn't move at the same rate as the top deck. The abutment footing is assumed to remain stationary although in actuality the diaphragm may be dragging the footing along.

Footing movements are also shown in Tables 4 through 10. A plus "+" sign indicates the distances between reference points increased. A minus "-" sign indicates the distances between reference points decreased.

Several footing measurements show the abutment diaphragm moving in a direction opposite to that of the top deck. This could be explained by considering the moments induced in the abutment diaphragm by the deflection of the bridge deck. See Figure 17.

Deck shortening measurements were taken by field construction personnel on the various projects through the State. These measurements were not corrected for temperature variations.

Figure 18 (Page 26) shows the two-variable least squares line, coefficient of correlation, and plots of all the deck shortening measurements made to date. Initial deck lengths were recorded at various times after the top deck was cast but before the structure was stressed. These measurements were usually taken within 10 days after the deck was cast. All subsequent measurements were compared to the initial readings.

Time on Falsework

The construction history of each structure was recorded. This included dates of casting the soffit, stems, and deck as well as dates of stressing and falsework release. Figures 19 (Page 27) and 20 (Page 28) are graphs of initial deflection versus the number of days from the soffit pour and deck pour until the falsework was released. These plots indicate a poor correlation between the length of time a structure is on falsework and the initial deflection. These plots however do not account for the effect of increasing Ec with additional time on falsework. A delay of stressing from the 12th day (normal) to the 42nd day will increase Ec about 12% for 8 sk. concrete.

DEFLECTION DIAGRAM

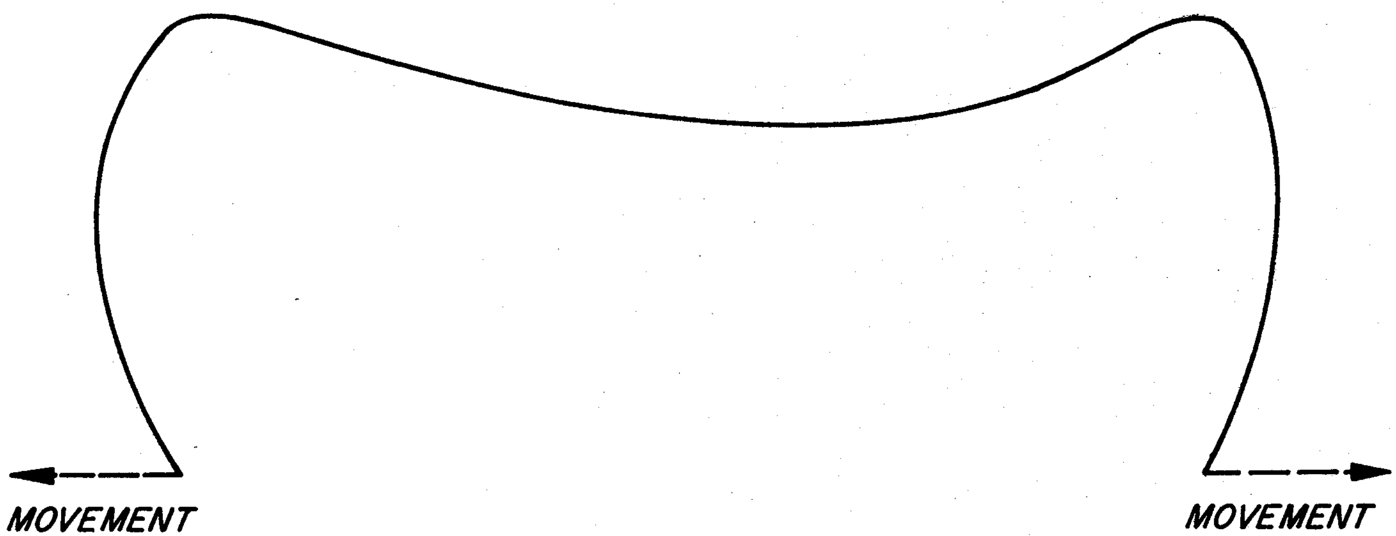


Figure 17

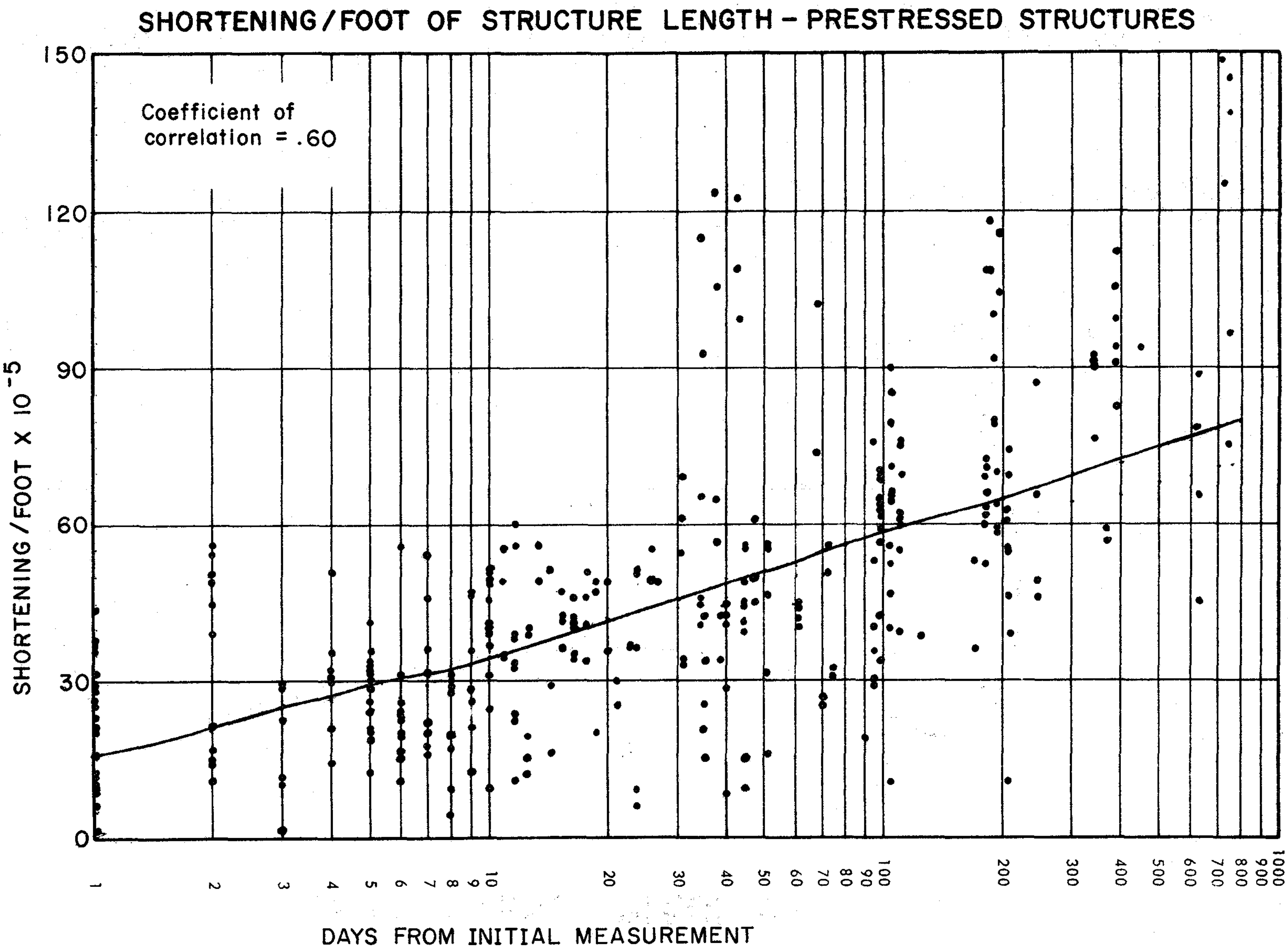

SHORTENING/FOOT OF STRUCTURE LENGTH - PRESTRESSED STRUCTURES
Coefficient of
correlation = .60
SHORTENING/FOOT X 10^-5
0
30
60
90
120
150
1
2
3
4
5
6
7
8
9
10
20
30
40
50
60
70
80
90
100
200
300
400
500
600
700
800
900
1000
DAYS FROM INITIAL MEASUREMENT

Figure 18

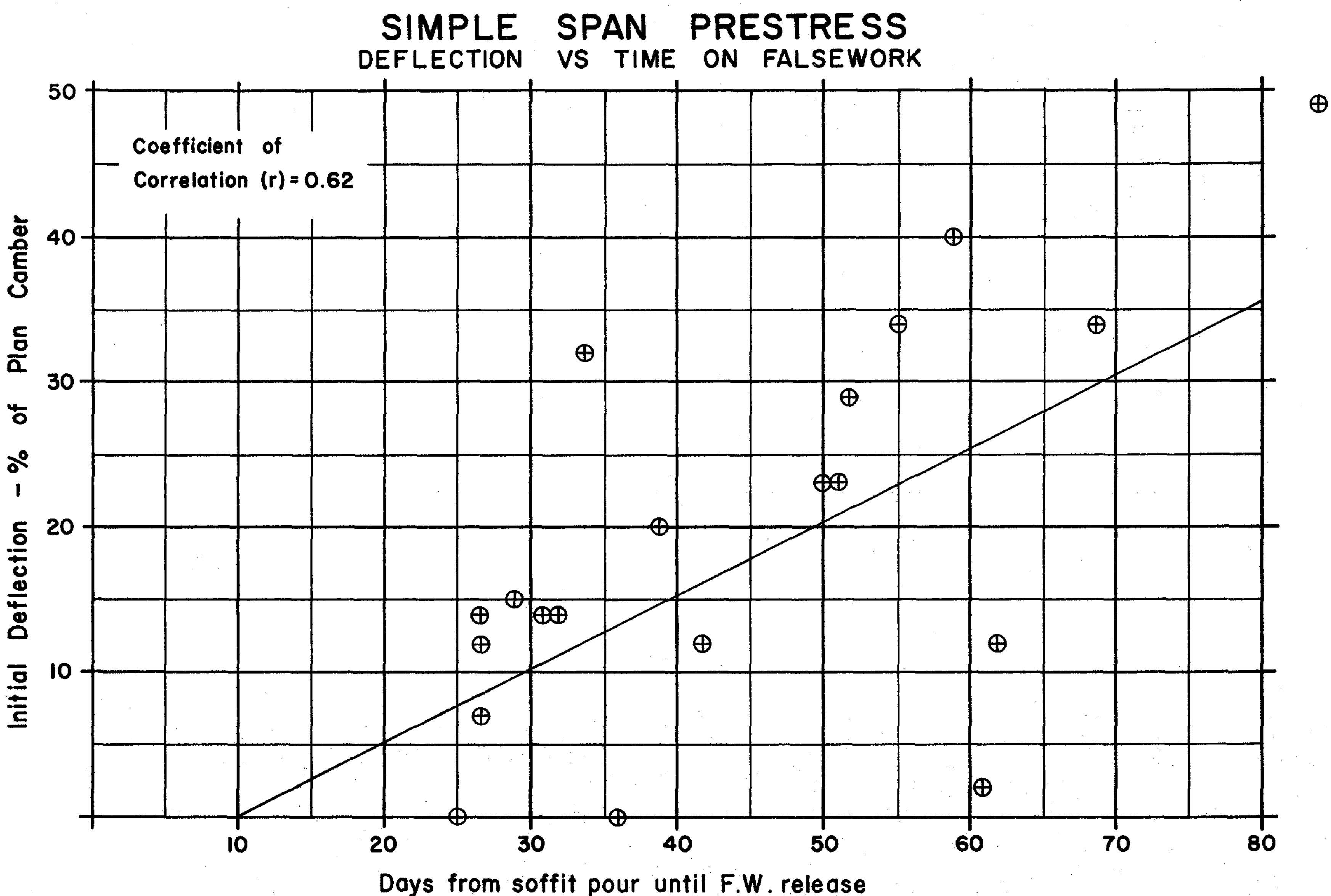

SIMPLE SPAN PRESTRESS
DEFLECTION VS TIME ON FALSEWORK
Coefficient of Correlation (r) = 0.62
Initial Deflection - % of Plan Camber
Days from soffit pour until F.W. release
10
20
30
40
50
60
70
80

Figure 19

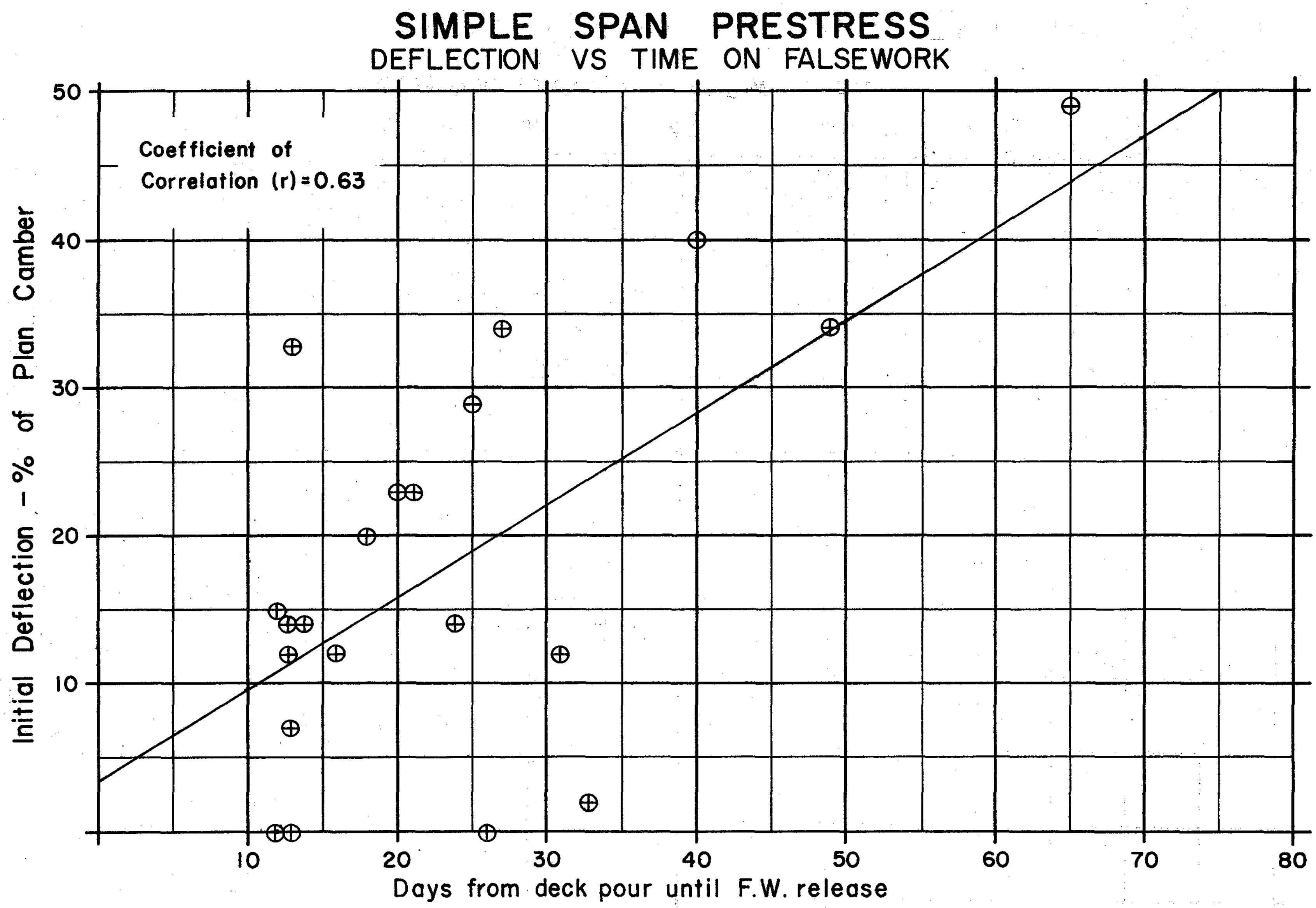

SIMPLE SPAN PRESTRESS
DEFLECTION VS TIME ON FALSEWORK
Coefficient of Correlation (r)=0.63
Initial Deflection - % of Plan Camber
Days from deck pour until F.W. release
10
20
30
40
50
60
70
80

Figure 20

Skew

The bridges used in the evaluation of the effect of skew on deflection were constructed between 1973 and 1976. Sample bridges were limited to widths where the top deck could be cast in one finishing operation. There were 13 bridges with significant skew in this study and a tentative skew correction formula was developed. (See Page 65.)

The California Department of Transportation, Division of Structures has (December 1975) made a finite element analysis of the effect of skew on bridge deflection.(1) This study is available upon request.

End Condition

Typical end supports for structures in this research project are shown in Figures 21 and 22.

Simple span structures may have both ends restrained from movement or have one end restrained and the other end free to slide. Continuous span structures have combinations of fixed and sliding supports. Tables 1-3 in the appendix list the type of footing for each structure in this project.

TYPICAL ABUTMENT SUPPORTS

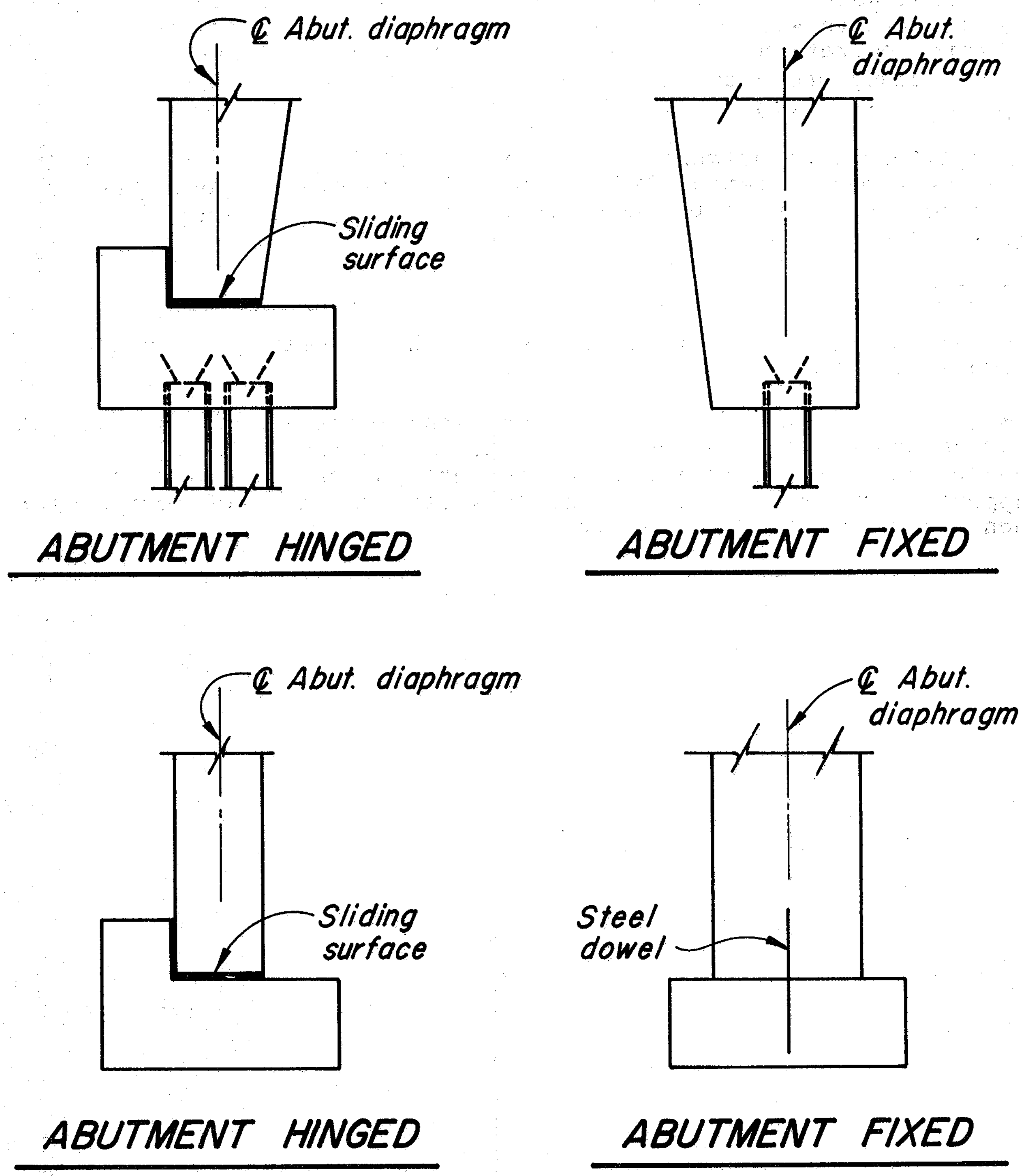
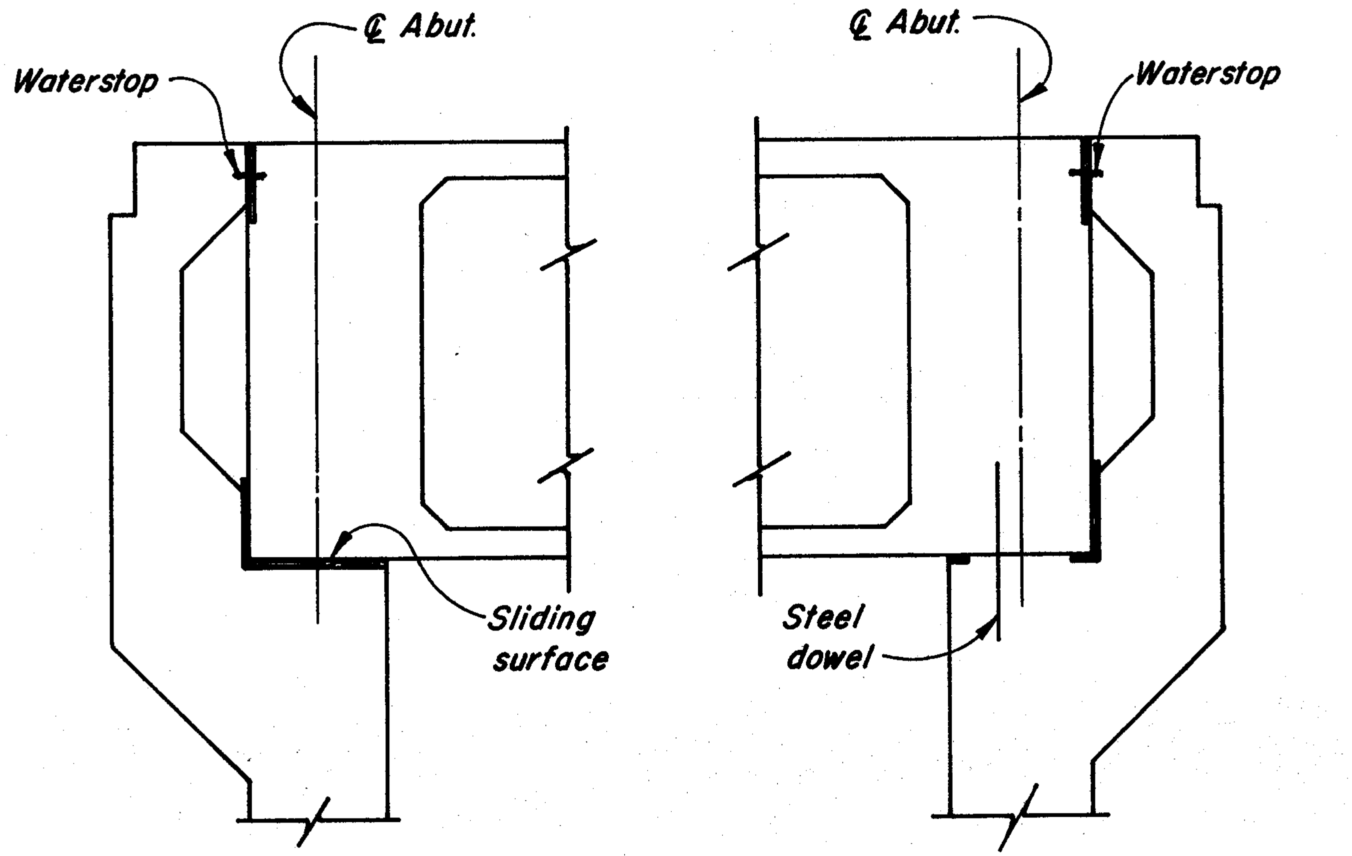


Figure 21

Figure 22

ABUTMENT SUPPORT BR. NO. 53-2373, 23-175, 23-173L &QL, 57-747



ABUTMENT HINGED

ABUTMENT FIXED

Falsework Load Changes

Calibration tests were made on the twelve Enerpac LH-5006, 100,000-pound (45,359Kg) capacity mini-hydraulic load cells at the California Department of Transportation Laboratory.

Results of these tests indicated maximum variations from the true readings as follows:

Load (lbs.)	(kg)	% Error
20,000	(9072)	6.4
40,000	(18144)	2.2
60,000	(27216)	1.7
80,000	(36287)	1.7
100,000	(45359)	1.5

The following data and plots were not adjusted for the possible load cell variations.

Periodic readings were taken on the cells which were placed under selected falsework posts at the following three bridges which are described in detail on pages 4 and 5.

1. Allendale Road Undercrossing, Bridge No. 23-169R

   Load cells were placed in falsework bents 3 and 4 as shown in Figures 26 and 27. A 12" x 12" x 3/8" (.3m x .3m x 9.5mm) steel distribution plate was placed on the top and bottom of each load cell. These plates prevented local overstressing of the timber falsework post bearing area.

   Falsework post settlement and load cell readings were made in the mornings at regular intervals. Additional readings were made in the afternoon after it was noticed that readings varied throughout the day.

   Bridge Chronology was as follows:

Cast girder stems and bottom slab	March 16, 1973
Cast top desk	April 6, 1973
Prestress bridge	April 16, 1973
Release falsework	April 19, 1973

2. M Street Undercrossing, Bridge No. 42-228R

   Load cells with steel distribution plates were placed in bents 2 and 3, Figure 32. Load cell readings and falsework post elevations were taken at regular intervals during the pour operation and on a daily basis until the falsework was removed.

Load cell readings taken following the stem pour indicated substantial increases (25-30%) between the morning and afternoon readings.

Bridge chronology was as follows:

Cast girder stems and bottom slab	May 31, 1973
Cast top deck slab	July 20, 1973
Prestress bridge	August 7, 1973
Release falsework	August 13, 1973

3. Old Oregon Trail Undercrossing, Bridge No. 6-178L

Load cells and bearing plates were placed under falsework posts in bents 3, 4, and 5 as shown in Figure 36. Load cell and settlement readings were recorded from bottom slab and stem pour until after the structure was prestressed.

Bridge chronology was as follows:

Cast bottom slab and girder stems	August 30, 1974
Cast top deck slab	September 17, 1974
Prestress bridge	September 30, 1974
Release falsework	October 5, 1974

Enerpac LH-5006

Hydraulic Load Cells

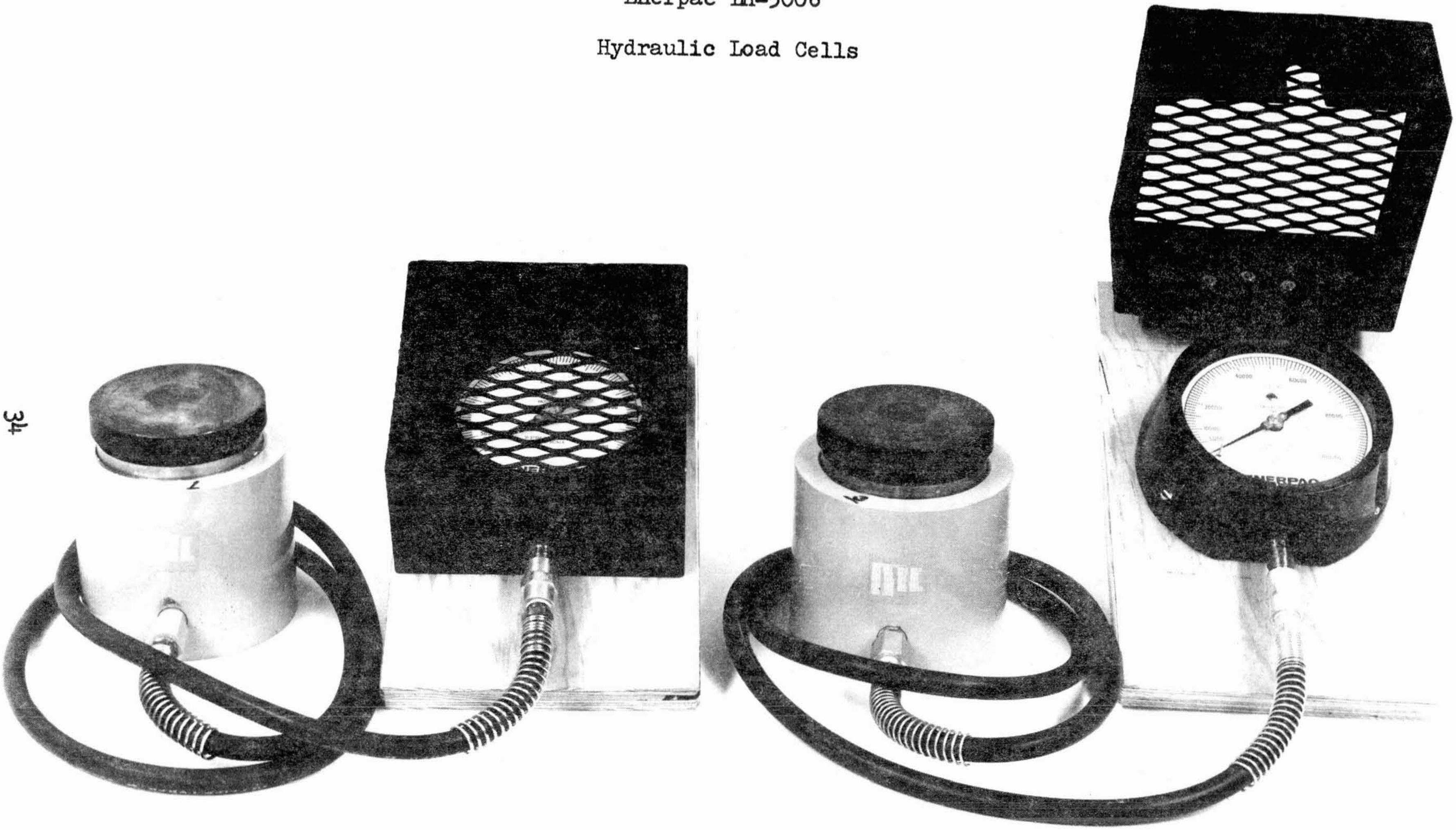
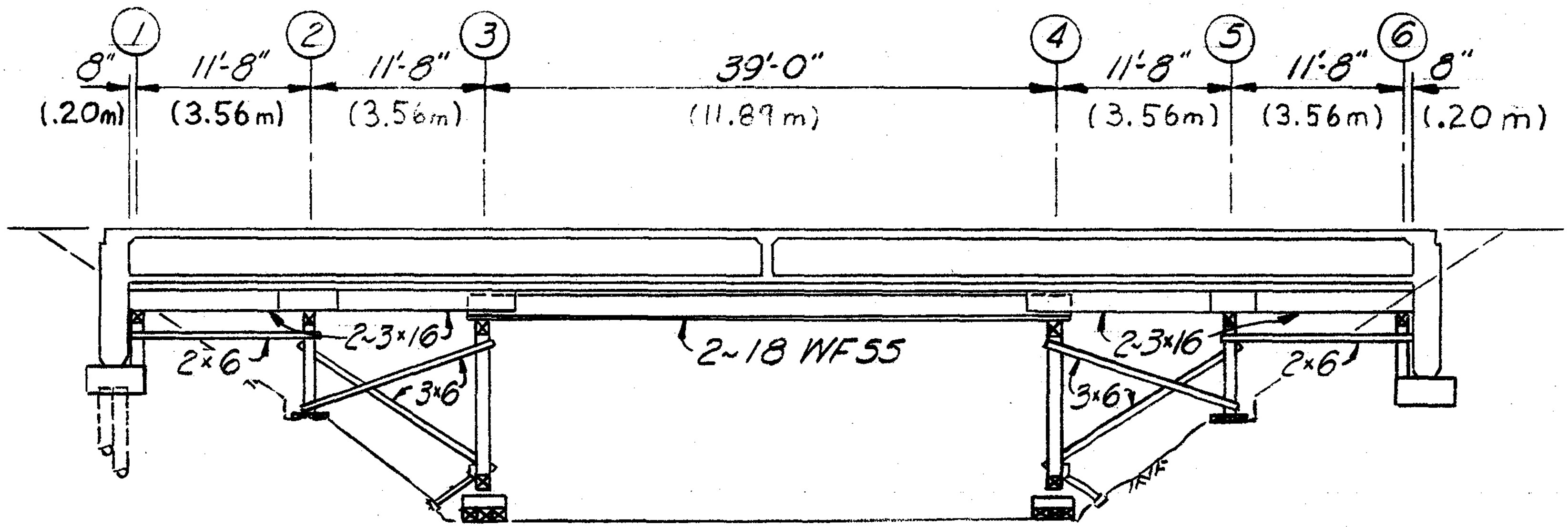


Figure 23



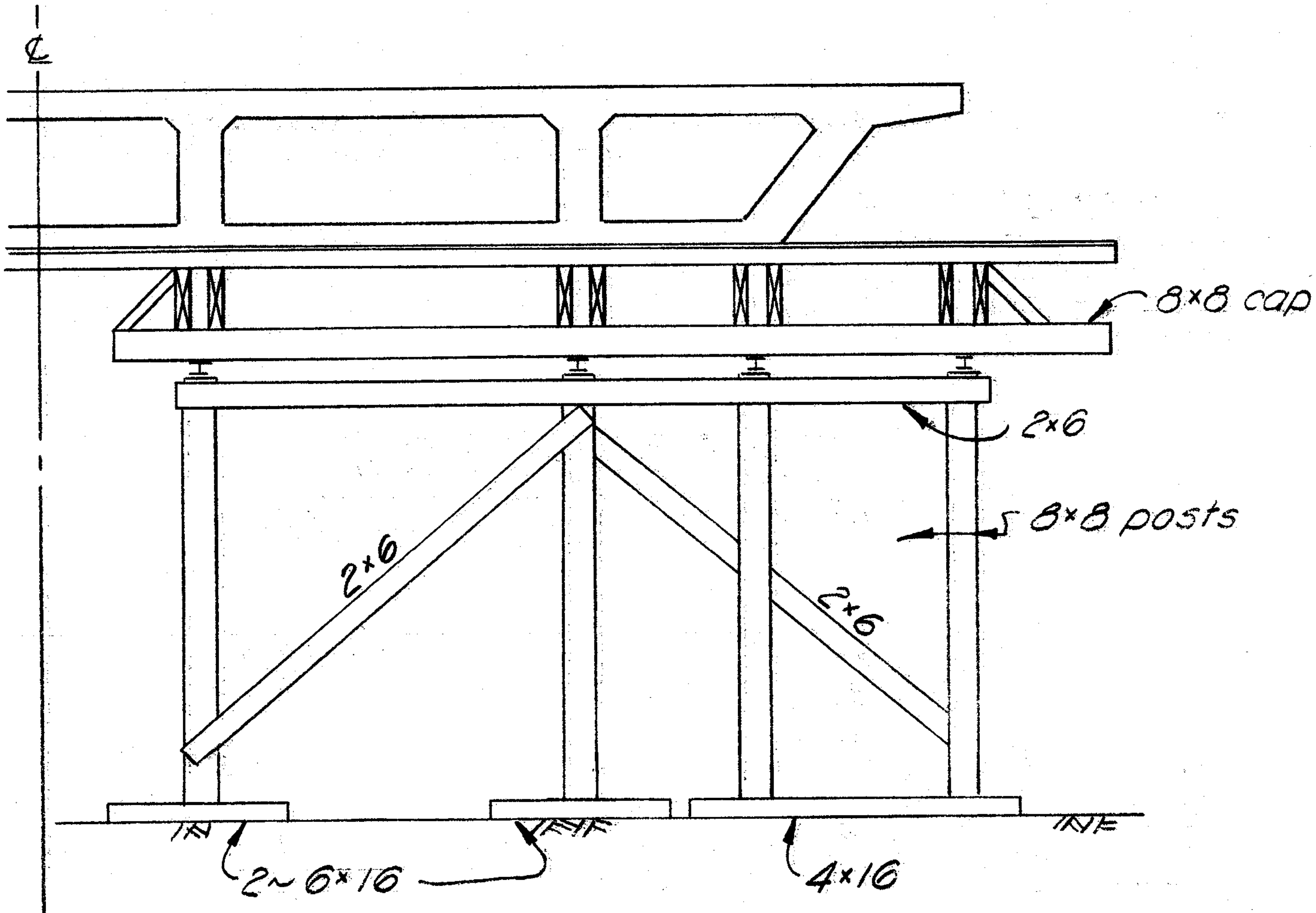
LONGITUDINAL SECTION

FALSEWORK LAYOUT

ALLENDALE ROAD U.C.

Br. No. 23-169 R

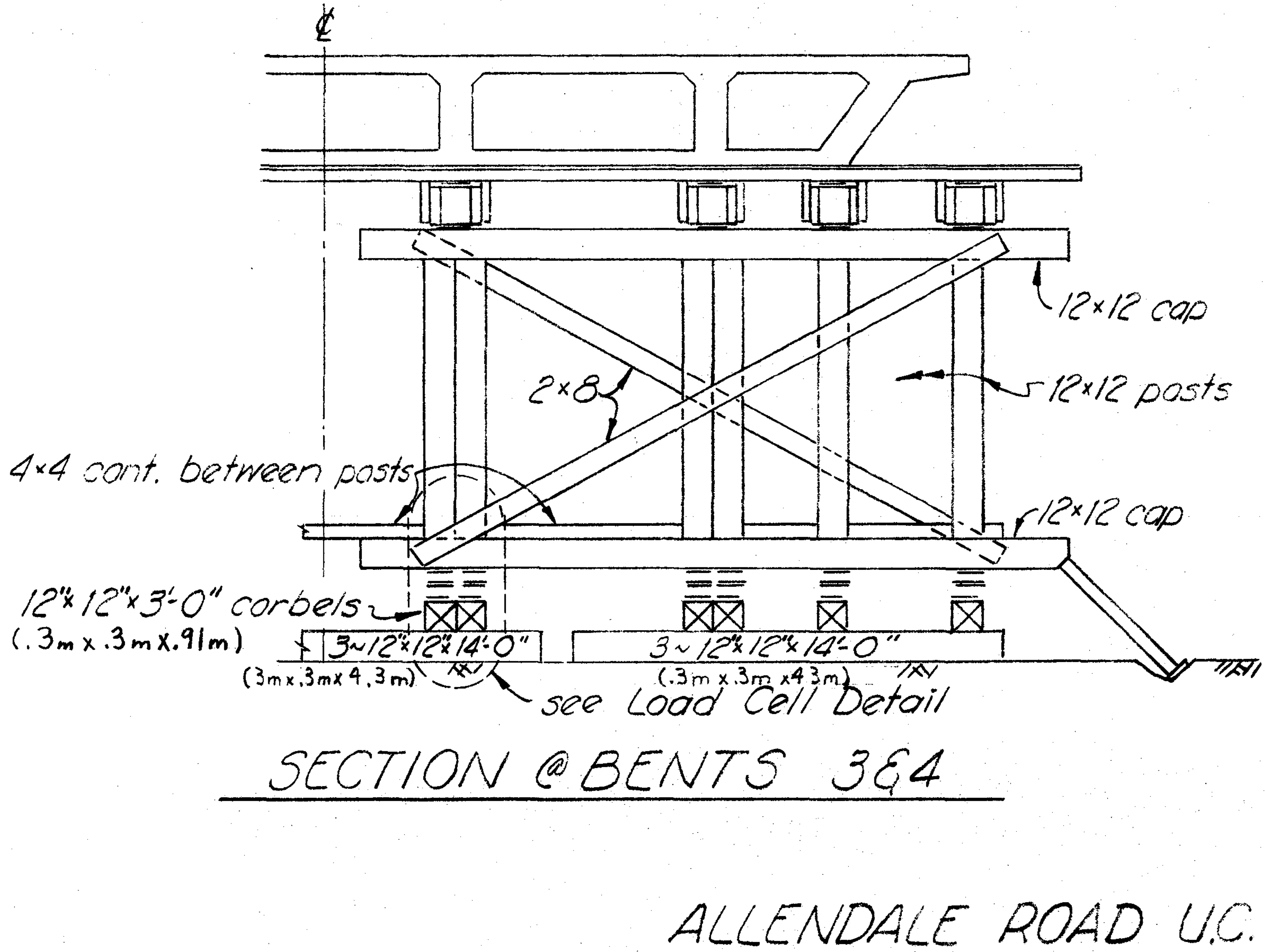
Figure 24



SECTION @ BENTS 2&5

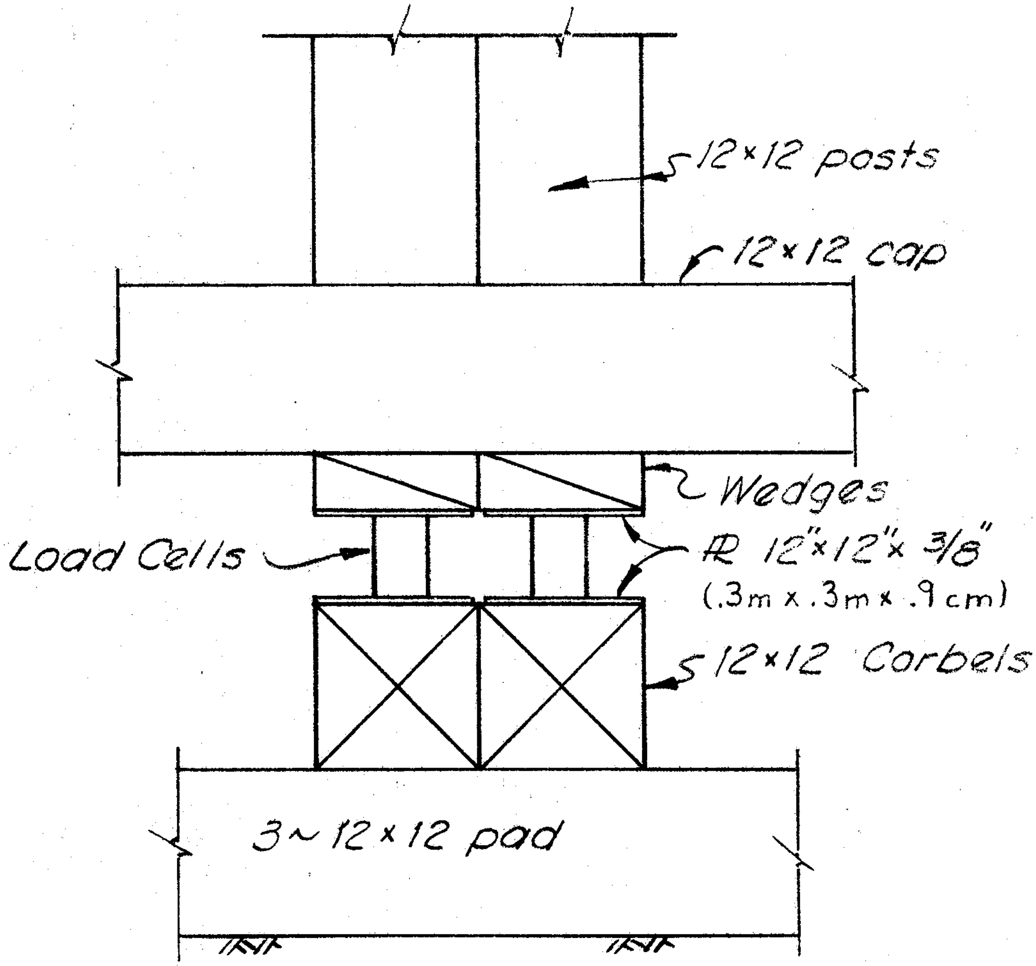
ALLENDALE ROAD U.C.

Figure 25



℄
12×12 cap
12×12 posts
2×8
4×4 cont. between posts
12×12 cap
12"×12"×3'-0" corbels
(.3m x .3m x .91m)
3~12"×12"×14'-0"
(3m x .3m x 4.3m)
3~12"×12"×14'-0"
(.3m x .3m x 4.3m)
see Load Cell Detail
SECTION @ BENTS 3&4
ALLENDALE ROAD U.C.

Figure 26

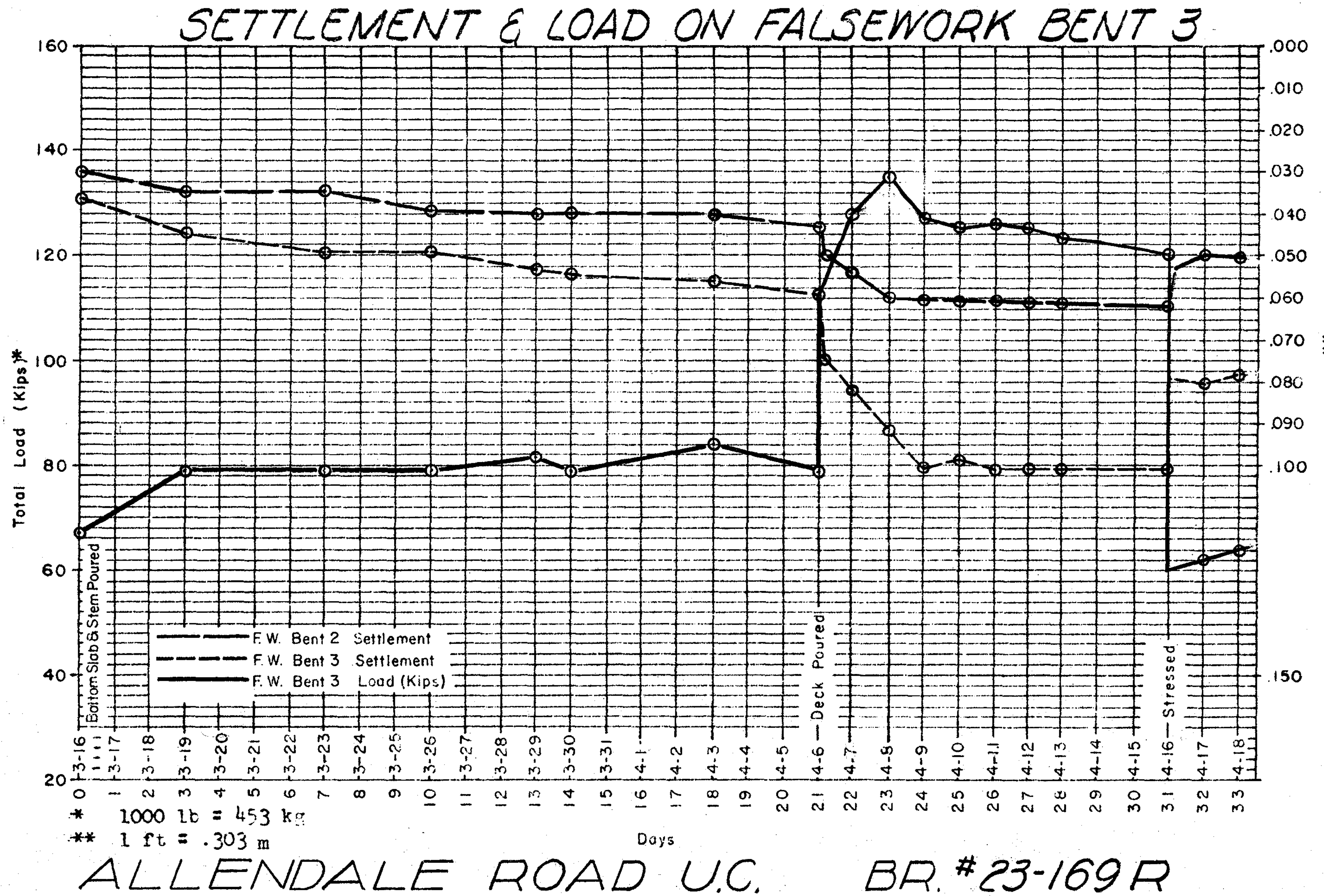


LOAD CELL DETAIL

Double post shown

ALLENDALE ROAD U.C.

Figure 27



SETTLEMENT & LOAD ON FALSEWORK BENT 3
Total Load (Kips)*
Settlement (ft.) **
160
140
120
100
80
60
40
20
.000
.010
.020
.030
.040
.050
.060
.070
.080
.090
.100
.150
Bottom Slab & Stem Poured
Deck Poured
Stressed
F. W. Bent 2 Settlement
F. W. Bent 3 Settlement
F. W. Bent 3 Load (Kips)
3-16
3-17
3-18
3-19
3-20
3-21
3-22
3-23
3-24
3-25
3-26
3-27
3-28
3-29
3-30
3-31
4-1
4-2
4-3
4-4
4-5
4-6
4-7
4-8
4-9
4-10
4-11
4-12
4-13
4-14
4-15
4-16
4-17
4-18
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33
Days
* 1000 lb = 453 kg
** 1 ft = .303 m
ALLENDALE ROAD U.C. BR. #23-169R

Figure 28

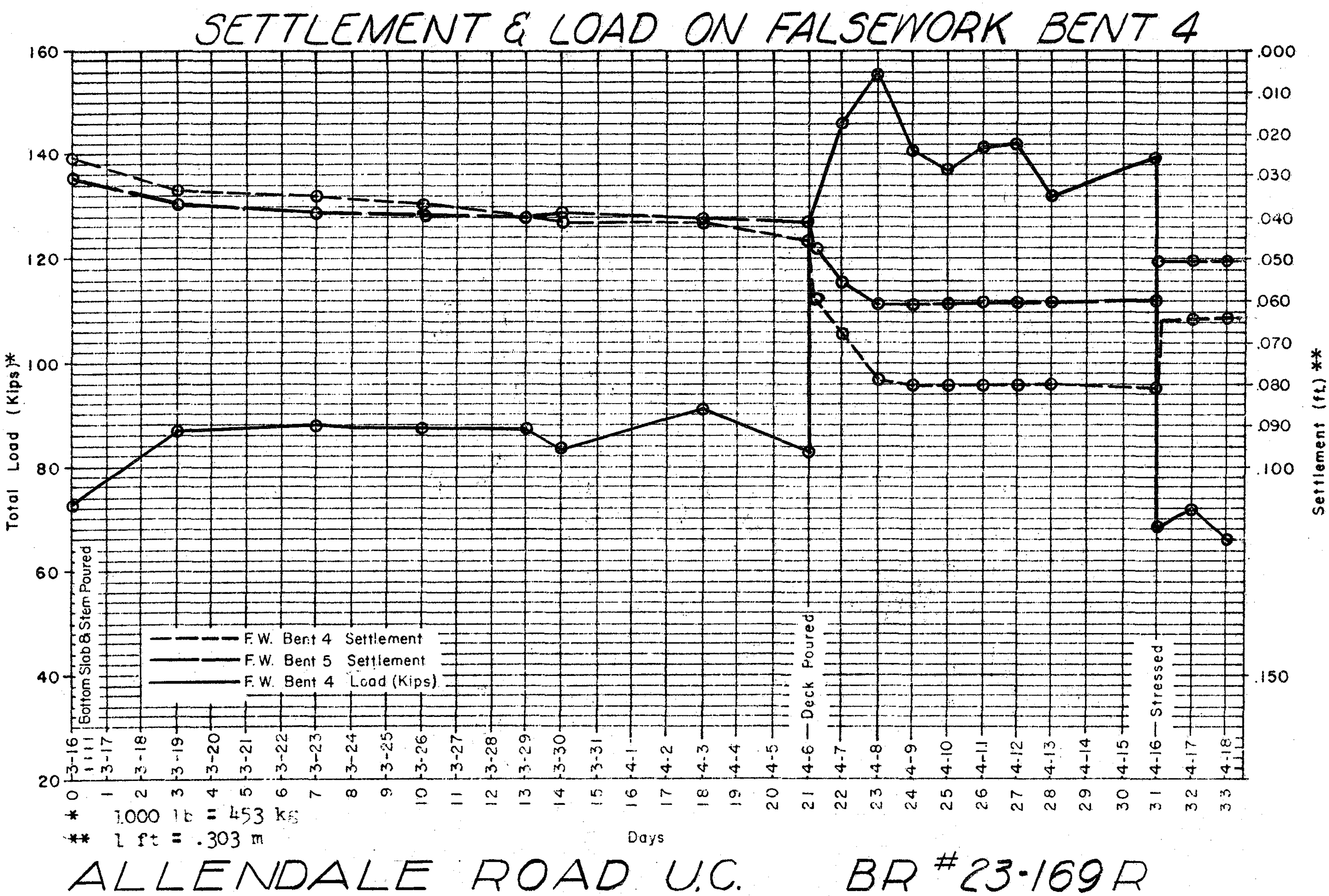
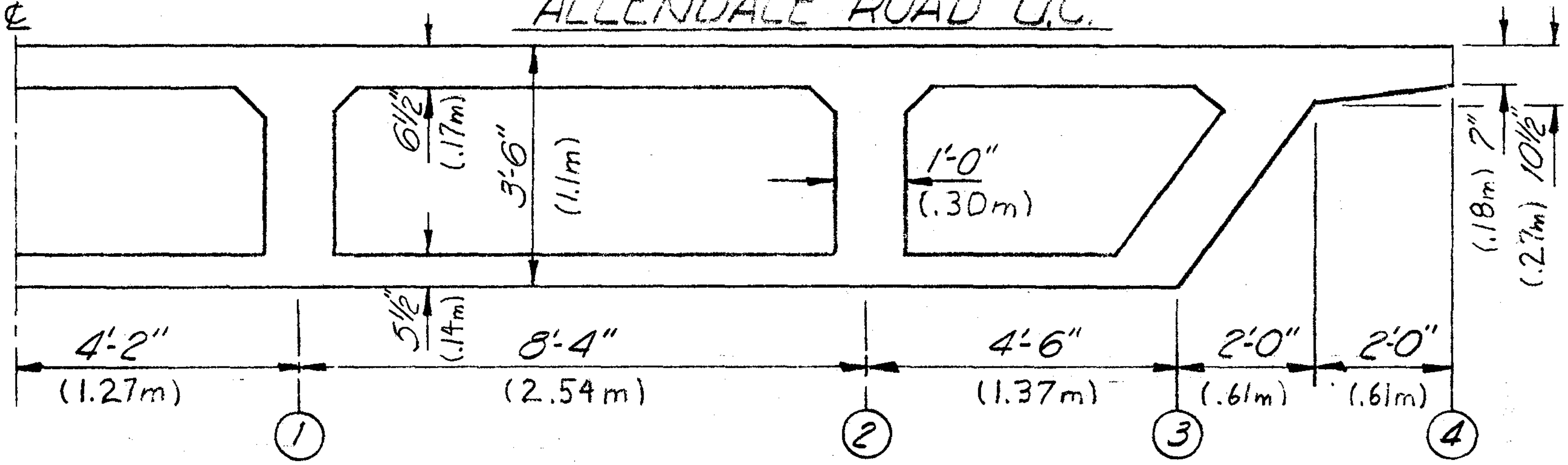

SETTLEMENT & LOAD ON FALSEWORK BENT 4
Total Load (Kips)*
Settlement (ft) **
F. W. Bent 4 Settlement
F. W. Bent 5 Settlement
F. W. Bent 4 Load (Kips)
Bottom Slab & Stem Poured
Deck Poured
Stressed
Days
* 1000 lb = 453 kg
** 1 ft = .303 m
ALLENDALE ROAD U.C. BR #23-169R

Figure 29

COMPARISON OF MEASURED & CALCULATED FALSEWORD LOAD

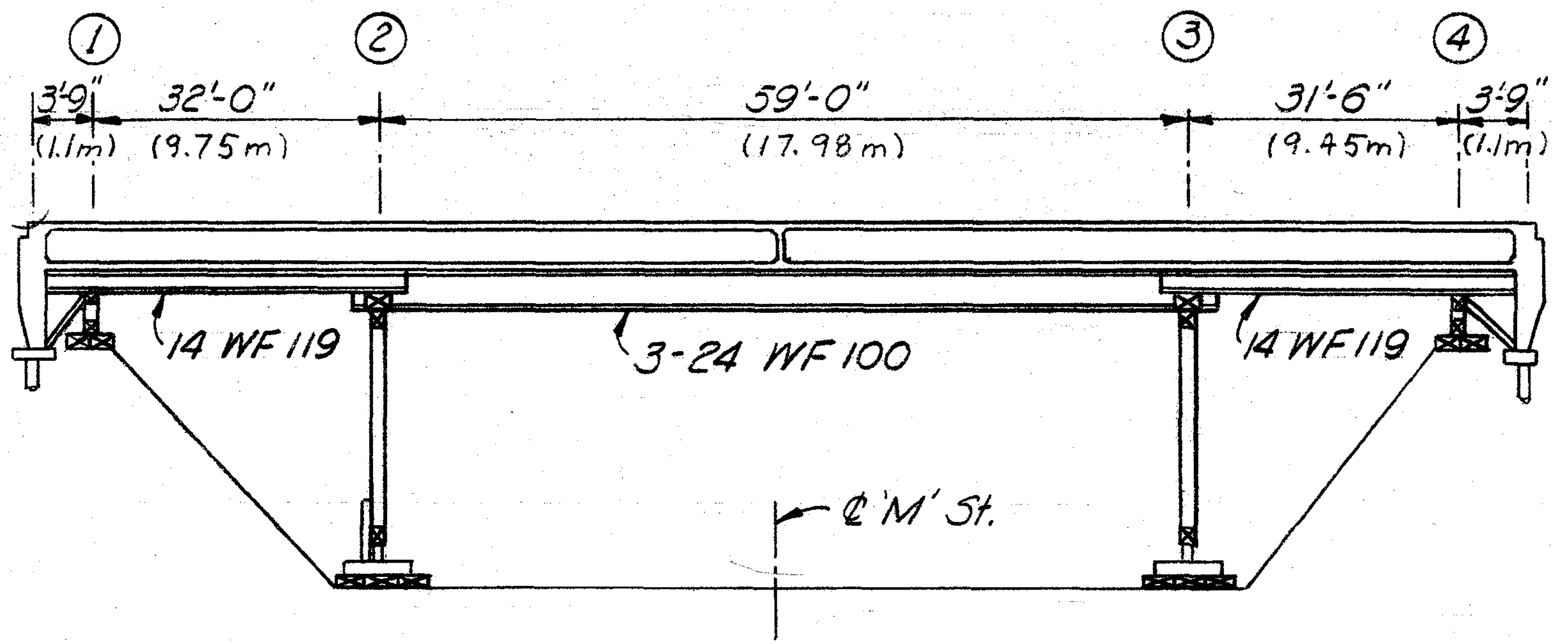
ALLENDALE ROAD U.C.



		* 1 (lb)	* 2 (lb)	* 3 (lb)	* 4 (lb)	* Total (lb)
Prior to Pour	Measured	8,000	4,500	6,000	7,000	25,500
Stems and Bottom Slab	Calculated	29,305	25,525	16,230	4,051	75,111
	Measured	31,500	20,000	7,000	14,500	73,000
Top Slab (total section)	Calculated	47,980	40,000	25,500	9,795	123,275
	Measured @ Pour	48,500	35,000	17,500	22,500	123,500
	Measured @ max. load	57,500	48,000	25,500	25,000	156,000

* 1000 lb = 453 kg

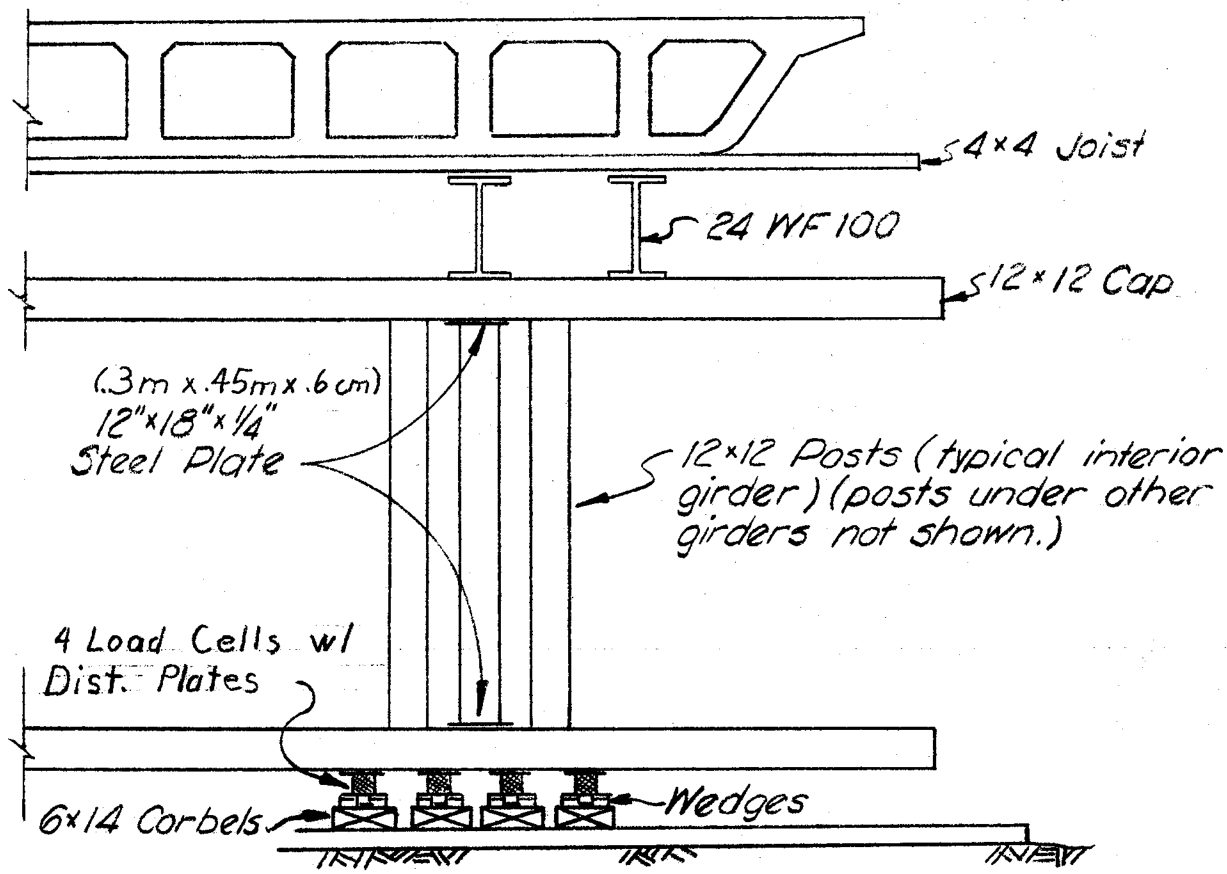
Figure 30



LONGITUDINAL SECTION

M STREET UC
Br. #42-228 R

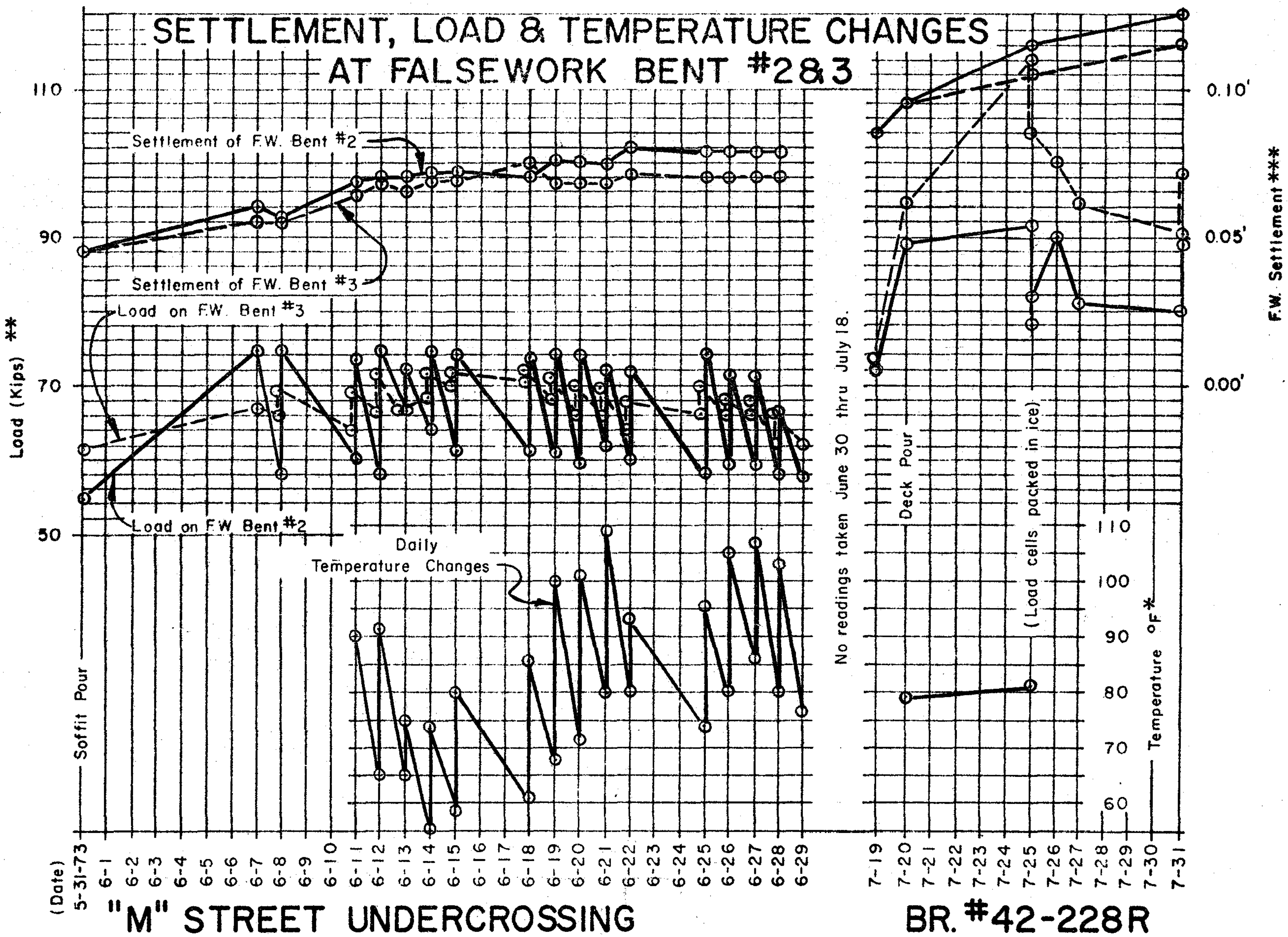
Figure 31



SECTION @ BENTS 2 & 3

M STREET UC
Br. #42-228 R

Figure 32

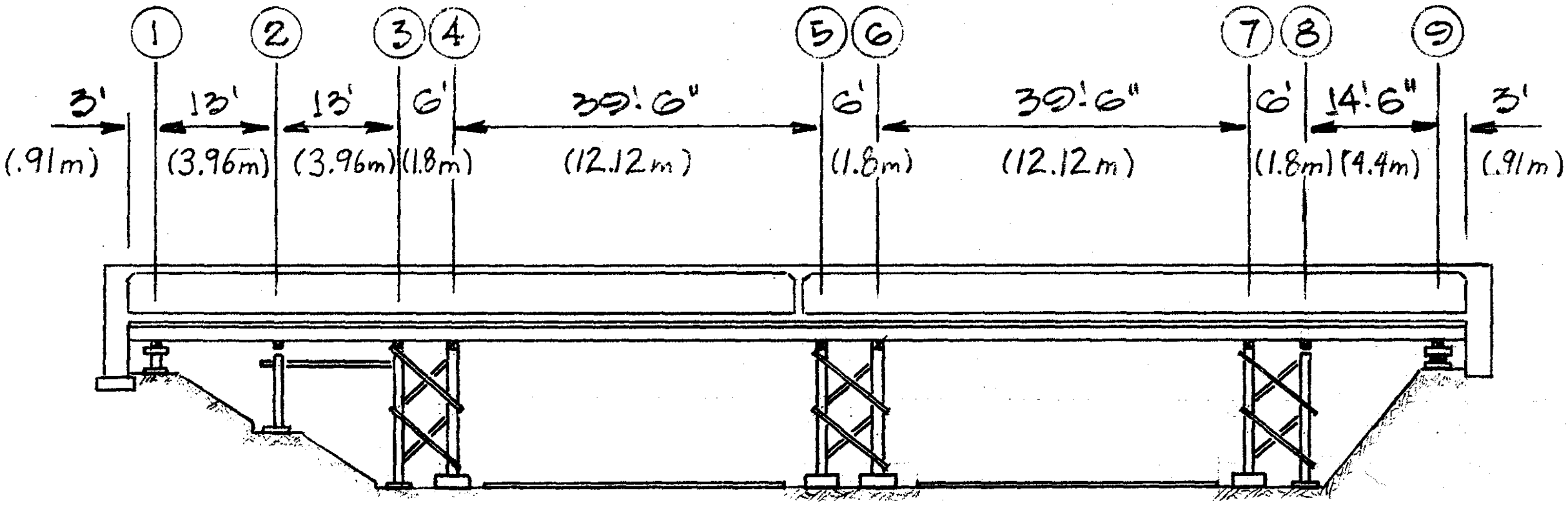


* °C = (°F - 32)/1.8

** 1000 lb = 453 kg

*** 1 ft = .303 m

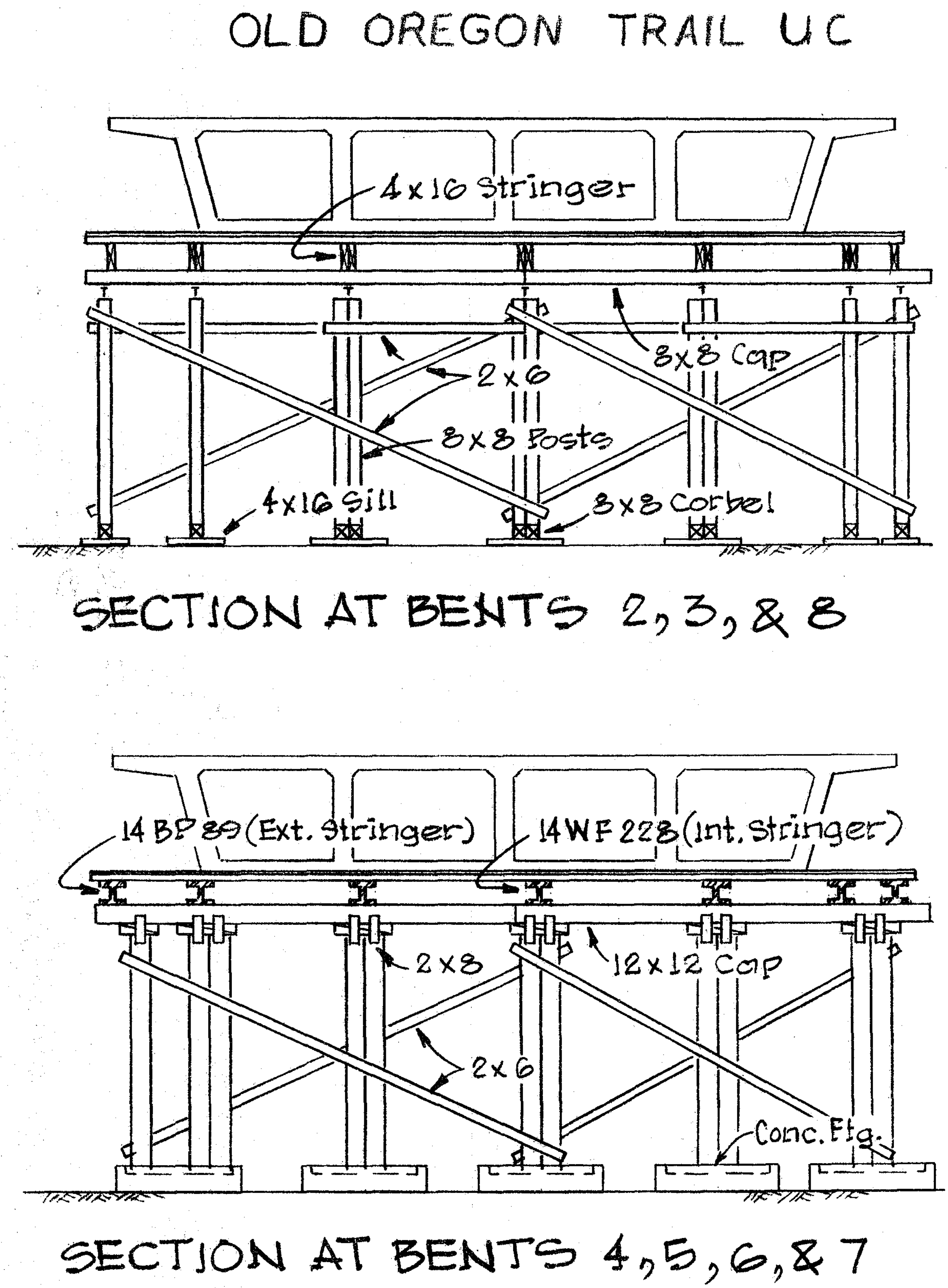
Figure 33



LONGITUDINAL SECTION
FALSEWORK LAYOUT

OLD OREGON TRAIL U.C.
Br. No. 6-178 L

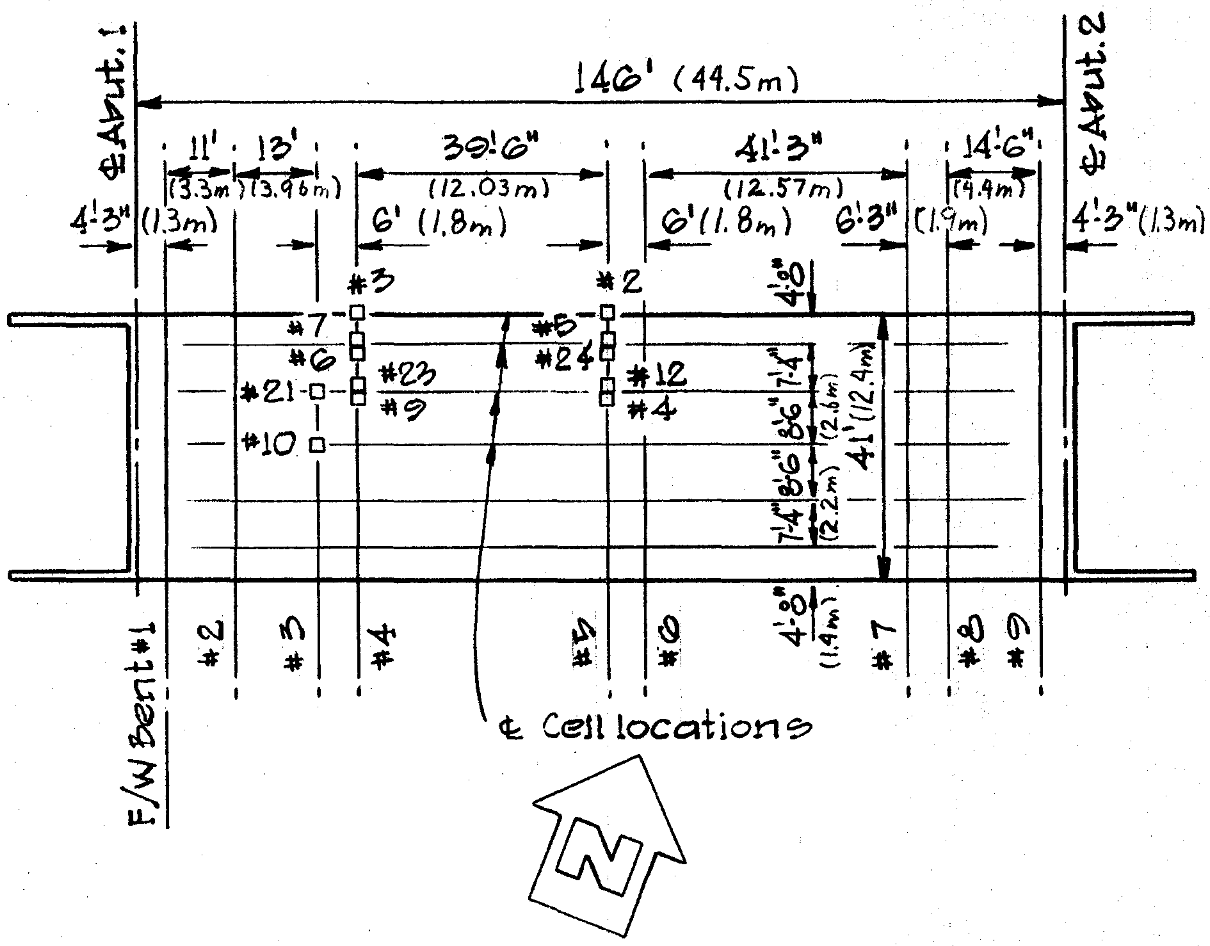
Figure 34



OLD OREGON TRAIL UC
4x16 Stringer
8x8 Cap
2x6
8x8 Posts
4x16 Sill
8x8 Corbel
SECTION AT BENTS 2, 3, & 8
14 BP 89 (Ext. Stringer)
14 WF 228 (Int. Stringer)
2x8
12x12 Cap
2x6
Conc. Ftg.
SECTION AT BENTS 4, 5, 6, & 7

Figure 35

FALSEWORK LOAD STUDY



OLD OREGON TRAIL U.C. (LT. BR.)
Br. No. 6-178L

Figure 36

Results

1. Allendale Road Undercrossing

Several inches of rain fell from the time the falsework erection started until concrete placement began. The ground around the falsework pads was quite wet.

Variations in morning and afternoon load cell readings were noted. Similar changes were also observed on the other two bridges in this study. A possible explanation is offered under "Discussion of Test Results".

A comparison of calculated loads and actual loads was made, Figure 30. Although individual girder loads varied the total loads at the bents were within .2% of those calculated. Calculated loads are based on a reinforced concrete weight of 160 lb/cf (2562Kg/m^3). A unit weight sample, taken during the stem and bottom slab pour, showed the actual weight of the concrete alone to be 150.5 lb/cf (2410.6 Kg/m^3).

Load cell readings increased 23% two days after the deck pour. The adjacent falsework bents settled during this time. These increased loads decreased as the settlement stabilized, Figures 28 and 29.

2. M Street Undercrossing

No rain fell while the falsework was in place. The summer temperatures varied from 55°F (13°C) in the morning to 108°F (45°C) in the afternoon.

Load cell readings increased consistently and as much as 28%, between the morning and afternoon. The maximum daily temperature change between readings was 32°F and on this day the load cell readings varied 21%.

Load cell readings are shown in Tables 11 through 23 (pages A11-A23).

Figure 33 shows a 21% increase in falsework loads by five days after the top deck was cast.

3. Old Oregon Trail

Load cell and settlement readings were taken by the Engineer on the job as the work schedule permitted.

A comparison of calculated versus measured falsework loads during various stages of construction is shown in Table 24 (page A24).

Load cell readings are shown in Tables 25 through 36 (pages A25-A36). These readings are plotted in Figures 37 through 40 (pages A37-A40). Figure 38 (page A46) indicates that the load on cell #12 increased 18.4% several days after the top deck was cast.

Discussion of Test Results

Various theories have been proposed to explain the daily fluctuation in load cell readings. When these fluctuations were noted during the construction of M Street, it was assumed that the fluctuations were due to significant temperature changes in the load cell. To test this explanation, the following procedure was taken.

Ice was packed around several load cells to see if the readings could be affected. Tables 19 (page A19) and 20 (page A20) indicate that readings did change with the application of ice. These load cells were later sent to Sacramento for testing to see how temperature changes influenced the readings. Tests made at the Transportation Laboratory indicated temperature changes had no significant effect on load readings. Discussions with the field engineer and observations of the testing in the laboratory leads to the following possible explanation.

The load cells in the field are under a sustained load. When ice was packed around the load cell the metal distribution plates and load cell itself cooled and contracted slightly. This contraction would relieve the load on the cell slightly and cause a lower reading.

The laboratory tests tried to duplicate the field conditions, but one significant difference was noted. In the laboratory tests, the load cells were either cooled in a refrigerator without load or heated in an oven without load and then placed in the testing machine. Loads were applied at 10,000 lb. (4535Kg) increments from 0 to 100,000 lbs. (45350Kg). These loads were applied at temperature ranges of 25°F± (-4°C±), 68°F± (20°C±), and 123°F± (50°C±).

This laboratory test loading procedure did not include a preloading from a restrained, stiff source and could, therefore, explain the inability to duplicate the field variations in the laboratory tests.

Field readings were taken without ice throughout the test period to provide consistency of data.

Another proposed explanation for the daily fluctuations is that the sun would heat the top deck surface faster than the rest of the box girder. This would cause the top deck to expand faster than the rest of the bridge. The simple span bridge would then hunch up and relieve the stress in the load cells if they were at mid-span and could increase the load of any cells placed between abutments and about 1/4 span points. This explanation fails at "M" Street UC where the load cells should have unloaded with increasing temperature.

Whatever the reason for the daily fluctuations the load changes seem to be related to changes in air temperature as the cell reading fluctuations matched the daily temperature cycle.

Another point of interest is the phenomenon of having the loads on the falsework posts near the center of span increase for several days after the deck is cast. This cannot be explained by hydration of the fresh concrete deck. During the hydration process, the deck would expand and tend to relieve these falsework post stresses and load the end supports more.

Expansion of falsework posts that are wetted during the soffit and stem pour could explain the increase in load cell readings following the soffit and stem pour. However, this would not explain the increase following the deck pour because the falsework posts don't get wet once the soffit and stems are cast.

The most likely explanation seems to be that the falsework bents have differential settlement. The falsework posts on the abutment fill tend to settle more than posts founded on original ground. There are also variations in post settlement within each falsework bent. This would cause a redistribution of loads to adjacent falsework posts. Once the bridge is stressed and the load on the falsework is released the supporting soil rebounds and the posts rise as indicated by the graphs. Plots of settlement and rebound are shown in Figures 28, 29, 33 and 37 thru 40.

Deflection at Centerline of Span

The 40 deflection curves in the Appendix (A54-A93) show the centerline of span deflections of some 17 single span prestressed structures and one 3-span prestressed structure. Both sides of each deck are represented by separate curves.

The elevations were taken in accordance with "research procedure" discussed earlier.

It was found that often a survey would indicate that one of the abutments had settled relative to the other. Generally, a later survey confirmed true abutment settlement and occasionally it indicated a one-time surveying error. In all cases the centerline elevation was adjusted by one-half of the change in difference of the two abutments. (If one abutment settles .10' the centerline elevation is .05 lower even if there is no change in true deflection.) In addition, only the last foot (digit) of the elevation is shown on the plots (i.e. 1034.753 is shown as 4.753).

The plan cambers shown are as calculated by the individual designers using the procedure specified in Memo to Designers 11-37 (1970) (Pages A52-A53). These were all calculated in the early 1970s. Some designers modified E and/or creep factors based on their judgment. The design final elevation (orig. elev.-camber) and M/EI camber are also shown on each plot. This M/EI camber was determined recently using the current office of structures frame system computer program. This program solves for deflection using a M/EI analysis. The program assumes a prestress loss of 32 ksi and a total deflection of three times the initial calculated deflection. The M/EI method is classically correct for uncracked, homogeneous sections. All the structures in this study calculated to be uncracked under the dead load and calculated section properties with the prestress losses assumed to be 32 ksi.

After a review of field data a tentative basis of predicted deflection was determined as follows:

1. The M/EI final deflection was considered good.
2. The final deflection was considered to occur at 20,000 days (54.8 years).
3. The final deflection was assumed to be three times the initial deflection.
4. A straight line variation in deflection was assumed when plotted on semi-log paper. (Time on log base.)

Following are some of the factors which are usually considered to affect calculated deflections.

1. The structural section is assumed to be homogeneous concrete, but is, in fact, reinforced concrete.

   Possibly a transformed section would be more accurate. We should expect actual deflections to be somewhat less than that calculated. This seemed to be the case on this project.

No adjustments were made in the calculated deflections for this factor.

2. The Modulus of Elasticity is usually assumed to be $3.5x10^6$ psi.

For this project Ec was determined for all concrete samples submitted for testing and the results are plotted on Figures 10 through 16. For 7 sack concrete (the standard for top slab) the Ec curve plot had a coefficient of correlation of only 0.42. At 60 days (for example) Ec averaged $4.62x10^6$ but varied between $3.2x10^6$ and $6.4x10^6$. Although most data ended at 60 days, extrapolation of this curve indicates that Ec increases slowly with a guestimate of $5.5x10^6$ as an average upper limit.

The calculated deflections were adjusted by the ratio of average test cylinder Ec at age of stressing to assumed Ec (usually $3.5x10^6$ psi). This amounted to as much as -24.9% (Ec = 4.30) or +7.1% (Ec = 3.25).

The Ec of the top slab (7.0 to 8.5 sack concrete) was averaged with the Ec of the soffit and stem concrete (6.0 to 8.5 sack concrete) to obtain an Ec of the composite section.

3. Absorption of the aggregate.

Other researchers (2) report creep related to absorption of the aggregate. Recent Caltrans lab tests (3) indicate the following relationships.

Source	Combined Agg. Absorp.%	Modulus E PSIx10^6 Test Age, Days 20	28	60	Creep @ 77 days load, In./In.x10^{-5} Age at Loading 10	28	60	Shrinkage @ 77 days Load In./In.x10^{-5} Days Wet Cure 10	28	60
Granite	1.04	3.93	4.40	4.61	51	34	30	42	44	44
Kaiser	1.44	3.19	3.64	4.06	62	47	41	67	65	64
Conrock	1.65	2.88	3.30	3.52	59	53	47	63	58	56

Aggregate source clearly could have an important effect on deflection.

As aggregate shrinkage (A.S.) values had been determined for aggregates on many of the test structures, the deflection values were adjusted on the following basis.

$$\text{Defl. Value \% Adj.} = (\text{A.S.}-.048) \times 250)$$

This empirical formula was developed from creep test results in Reference 3. It sets an adjustment of 0.0% at .048 A.S. value and a judgement decision of a maximum of -4.5% (less deflection) at a A.S. value of .030.

4. The difference in ages between the soffit pour and the top slab pour.

   When a top slab is placed, it will shrink more and faster than the soffit and slab concrete. This greater shrinkage will act as a prestressing force through the top slab and will cause a negative moment in even a simple span.

   The greater the difference in age of the two pours, the greater expected negative moment caused by greater relative shrinkage of the top slab. This would increase actual deflections over calculated, although other factors can override the age factor.

   The time between pours varied from a minimum of 10 days to 49 days. The deflection values were adjusted on the following basis.

$$\text{Defl Value \% Adj.} = 25-[(\frac{100-T}{100})^2 x25]$$

   This empirical formula was developed to set 0.0% adjustment at T=0 days to a maximum of 25% at T=100 days. A number of formulas were attempted and this formula seemed to adjust the deflection best.

   The maximum adjustment used was an increase of 18.5% in deflection for a 49 day age of soffit concrete when the top deck was placed.

5. The age of the deck placement at the time of stressing.

   A significant increase in this age should result in smaller deflection values. The effect of older (stronger) concrete at stressing is essentially the same as an increase in Ec at the time of stressing which has already been evaluated so no additional adjustment in deflection is recommended.

6. The period of the time between stressing and falsework release.

   The structures in this test averaged 6.6 days between stressing and falsework release with 55% having falsework fully released by three days. Thirty-five percent of the structures were left stressed on falsework for 10 to 20 days.

   The generally held thinking among engineers is that initial and final deflection will be reduced if stressed structures are left on falsework for a significant time. The deflection data on this project, however, indicated no significant change in initial or final deflection with times of up to 20 days on falsework after stressing.

7. Structures on skewed abutments should have less deflection than calculated as skewed abutments have the effect of shortening the effective span length.

   Although the "skewed" length can easily be calculated (cos(skew)) and a new deflection calculated, the effect is not fully related to the span length as the tendons still are located along the original span length and the edges of the structure still span the original length. It is in the center area of the span that the structure bridges across on the shorter "skewed" span.

   It was decided that the following formula best fit the data and gave reasonable results.

$$\% \text{ Adj} = 100\text{x} \; \frac{1-[\cos \text{ skew}]^2}{2}$$

8. The greater the slump of the concrete the greater the final deflection.

   There were extensive slump notations on the records for the test cylinders on this project though there is some question as whether they all represent actual measurements or eyeball judgements.

   It was decided to assume 3 1/2" slump for the standard deflection calculation and to assume a change of 1% in the deflection for each 1/2" change in slump.

   The largest adjustment in calculated deflection was an increase of 4.0% at Long Beach Blvd. UC where the slumps were recorded as 5" and 6".

9. Wet weather should cause the top slab to expand and reduce the deflection.

Temperatures were occasionally recorded during surveys, but there were no clear indications of wet or dry decks so this factor was not considered on this project.

10. Sunny weather should cause the top slab to expand and reduce the deflection.

    Although temperatures were occasionally recorded during surveys, there was no clear indications of whether the temperature was significantly greater on the top slab so this factor was not considered in this project.

11. Lateral embankment pressure against the diaphragm abutments and piles below the superstructure could cause a negative moment which could reduce the deflection.

    Most of the project structures have deep diaphragm abutments located on the single row of piles at one or both of the abutments. These abutments ranged from 9.5 feet deep (5.0-foot superstructure) to 16.0 feet deep (8.0-foot superstructure).

    A careful comparison of deflections of structures with diaphragm abuts with those without, indicates that for this project there were no reduced deflections due to negative moments caused by embankment pressure.

12. The basic M/EI deflection computer program now used by the Office of Structures assumes a total prestress loss of 32 ksi.

    A separate research project underway in Caltrans has tentatively determined that the eight-year prestress loss is about 40 ksi, so prestress deflections were recalculated for some 17 structures assuming a stress loss of 40 ksi. The mean deflection increased 22% with a S.D. of 12%. Deflections calculated for a tentative full life stress loss of 50 ksi had a mean deflection increase of 58% with a S.D. of 25%.

    Due to the difficulty in assuming prestress loss for a particular structure at a particular time, this factor was not used in modifying the originally calculated M/EI deflections.

13. The basic M/EI deflection computer program assumes curb and rail dead loads at the time of stressing. This should give actual initial deflections slightly less than calculated.

    This factor was not used in modifying the originally calculated M/EI deflections.

Discussion of Deflection Plots

South "P" St. Rt. (Pages A46 & A47)

This structure did not deflect below cast elevation when the falsework was pulled. The calculated initial deflection was .03', but the structure actually was .01' higher after falsework was struck. The structure approached the calculated rate and amount of deflection by 200 days, but then rose significantly so that by 800 days it was higher than the original cast elevation. The reason for the uplift in the span after the first year is unknown. The M/EI deflection calculation gives a poor prediction of the actual deflection, but this is primarily caused by the unexplained late stage uplift in the span.

South "P" St. Lt. (Pages A48 & A49)

This structure did not deflect below cast elevation when the falsework was pulled. The calculated initial deflection was .03', but the structure actually was .01' higher after falsework was struck. The structure approached the calculated rate and amount of deflection by 127 days but then rose significantly so that by 400 days it was .03' higher than cast. The reason for the uplift in the span after the first 127 days is unknown. The M/EI deflection calculation gives a poor prediction of the actual deflection but this is primarily caused by the unexplained late stage uplift in the span.

Madison St. UC Rt. (Pages A50 & A51)

This structure did not deflect below cast elevation when the falsework was pulled, although its calculated initial deflection was .03'.

The comparison of this structure with its twin (below) is very interesting because its projected final deflection is only 40% of that calculated while the left Br. (twin) final deflection is 67% more than that calculated. The reasons for this significant difference is unknown as Ec, aggregate shrinkage, age of soffit concrete at top slab placement, skew and slump were all about the same. The M/EI deflection calculation gives a poor prediction (142% high) of projected final deflection.

Madison St. UC Lt. (Pages A52 & A53)

This structure deflected 63% of the adjusted calculated initial deflection when the falsework was pulled.

The comparison of this structure with its twin (above) is very interesting because of the factors listed above. The M/EI deflection calculation gives a poor prediction (40% low) of projected final deflection.

7th Ave UC Rt. (Pages A54 & A55)

This structure deflected 85% of the adjusted calculated initial deflection when the falsework was pulled.

This structure has a projected final deflection of 31% more than that calculated. The calculated deflection was decreased 23% due to an Ec of 4.46×10^6, increased 16% due to the soffit concrete being 41 days old when top slab placed and reduced 7.3% because of the 22° skew. The adjusted M/EI deflection calculation gives only a fair prediction (23% low) of projected final deflection.

7th Ave UC Lt. (Pages A56 & A57)

The left side of this structure deflected 7% more than the adjusted calculated initial deflection when the falsework was pulled.

The right side of this structure deflected 47% more than the adjusted calculated initial deflection when the falsework was pulled.

Both sides, however, have a projected final deflection of 9% more than that calculated. The calculated deflection was decreased 19% due to an Ec of 4.17×10^6, increased 16% due to the soffit concrete being 41 days old when top slab placed and reduced 7.3% because of 22° skew. The adjusted M/EI deflection calculation gives a good prediction (8% low) of projected final deflection.

Canyon Road UC Rt. (Pages A58 & A59)

This structure deflected only about 43% of calculated M/EI deflection (adjusted) when falsework was pulled. The structure quickly approached and maintained the calculated rate and amount of deflection through the last reading at 2,800 days.

The calculated deflection was decreased 25% due to an Ec of 4.37×10^6 and was increased 18% due to the soffit concrete being 49 days old when the top slab was placed.

The adjusted M/ET deflection calculation gives a good prediction (10% low) of projected final deflection.

Canyon Road UC Lt. (Pages A60 & A61)

This structure deflected only about 30% of calculated M/EI deflection (adjusted) when falsework was pulled. The structure quickly approached and maintained the calculated rate and amount of deflection through the last reading at 2,800 days.

The calculated deflection was decreased 5% due to an Ec of 3.67×10^6 and was increased 18% due to the soffit concrete being 49 days old when the top slab was placed.

The adjusted M/EI deflection calculation gives a good prediction (9% high) of projected final deflection.

Meyer Road UC Rt. (Pages A62 & A63)

The structure deflected only 28% of the calculated M/EI initial deflection (adjusted) when falsework was pulled.

The calculated deflection was decreased 7% due to an Ec of 3.7×10^6, increased 6.5% due to the soffit concrete being 14 days old when the top slab was placed and decreased 30% due to a 45° skew.

The adjusted M/EI deflection calculation gave a fair prediction (17% high) of projected final deflection.

Meyer Road UC Lt. (Pages A64 & A65)

This structure deflected about 87% of the calculated M/EI initial deflection (adjusted) when falsework was pulled.

The calculated deflection was decreased 10% due to an Ec of 3.8×10^6, increased 6.5% due to the soffit concrete being 14 days old when the top slab was placed and decreased 25% due to a 45° skew.

The adjusted M/EI deflection calculation gave a good prediction (7% low) of projected final deflection.

Larkin Valley Road UC Lt. (Pages A66 & A67)

This structure deflected about 72% of the calculated M/EI initial deflection (adjusted) when falsework was pulled.

The calculated deflection was decreased 14% due to an Ec of 4.0×10^6, increased 8% due to the soffit concrete being 17 days old when the top slab was placed and decreased 16% due to a 32° skew.

The adjusted M/EI deflection calculation gave a fair prediction (8% low) of projected final deflection.

Division St. UC (Pages A68 & A69)

This structure deflected about 112% of the calculated M/EI deflection (adjusted) when falsework was pulled.

The calculated deflection was decreased 20% due to an Ec of 4.2×10^6, decreased 5% due to low aggregate shrinkage; increased 12% due to the soffit concrete being 27 days old when the top slab was placed and decreased 8% due to a 15° skew.

The adjusted M/EI deflection calculation gave a fair prediction (22% high) of projected final deflection.

Susana Road UC (Pages A70 & A71)

This structure deflected about 84% of the calculated M/EI deflection (adjusted) when the falsework was pulled.

The calculated deflection was increased 7% due to an Ec of 3.3×10^6, increased 12% due to the soffit concrete being 27 days old when the top slab was placed and decreased 7% due to a 13° skew.

The adjusted M/EI deflection calculation gave a fair prediction (17% high) of the projected final deflection.

San Ysidro Blvd UC Rt. (Pages A72 & A73)

This structure only deflected about 8% of the calculated M/EI deflection (adjusted) when the falsework was pulled.

Although there were no Ec tests on this structure the calculated deflection was decreased 15% on the assumption that the concrete probably had an Ec of 4.0×10^6 (the same as the left structure). In addition, calculated deflection was decreased 4% due to the low aggregate shrinkage, increased 12% due to the soffit concrete being 28 days old when the top slab was placed, decreased 13% due to a 25° skew and increased 4% due to 5 1/2" slump.

The adjusted M/EI deflection calculation gave a fair prediction (13% low) of projected final deflection.

San Ysidro Blvd UC Lt. (Pages A74 & A75)

This structure deflected about 165% of the calculated M/EI deflection (adjusted) when the falsework was pulled.

The calculated deflection was decreased 15% due to an Ec of 4.0×10^6, decreased 4% due to the low aggregate shrinkage, increased 12% due to the soffit concrete being 28 days old when the top slab was placed and decreased 9% due to a 18° skew.

The adjusted M/EI deflection calculation gave a poor prediction (34% low) of the projected final deflection.

Long Beach Blvd UC (Left Half) (Pages A76 & A77)

This structure deflected 127% of the calculated M/EI deflection (adjusted) when the falsework was pulled.

Due to a six week long labor strike the deck was an unusual 60 days old at the time of stressing.

The calculated deflection was decreased 20% due to an Ec of 4.2×10^6, increased 8% due to the soffit concrete being 17 days old when the top slab was placed, decreased 7% due to a 14° skew and increased 4% due to a 5 1/2" slump.

The adjusted M/EI deflection calculation gave a fair prediction (11% high) of the projected final deflection.

Long Beach Blvd UC (Right Half) (Pages A78 & A79)

This structure deflected 92% of the calculated M/EI deflection (adjusted) when the falsework was pulled.

The calculated deflection was decreased 1.4% due to an Ec of $3.55x10^6$, increased 9% due to soffit concrete being 20 days old when the top slab was placed and decreased 7% due to a 14° skew.

The adjusted M/EI deflection calculation gave a fair prediction (12% low) of the projected final deflection.

Market St Off-Ramp OC (Pages A80 & A85)

This is a 3 span continuous structure.

Span #1

The center of span rose 178% of the calculated M/EI deflection (adjusted) when the falsework was pulled.

The calculated deflection (rise in span) was increased 23% due to an Ec of $4.3x10^6$, increased 4% due to low aggregate shrinkage and decreased 6% due to the soffit concrete being 12 days old when the top slab was poured.

The adjusted M/EI deflection (rise) calculation gave a good prediction (9% high) of the projected final deflection (rise).

Span #2

The structure deflected only 10% of the calculated M/EI deflection (adjusted) when the falsework was pulled.

The calculated deflection was decreased 16% due to an Ec of $4.1x10^6$, decreased 4% due to low aggregate shrinkage and increased 9% due to the soffit concrete being 19 days old when the top slab was poured.

The adjusted M/EI deflection calculation gave an excellent prediction (3% low) of the projected final deflection.

Span #3

The center of span rose 135% of the calculated M/EI deflection (adjusted) when the falsework was pulled.

The calculated deflection was increased 15% due to an Ec of $4.0x10^6$, increased 4% due to low aggregate shrinkage and decreased 7% due to the soffit concrete being 14 days old when the top slab was placed.

The adjusted M/EI deflection (rise) calculation gave a fair prediction (11% high) of the projected final deflection.

CONCLUSIONS

Bridge Deflections

Deflection data gathered so far tends to confirm the following generally held conclusions:

- Most structures rise off the falsework at midspan when prestressed.
- Most structures deflect downward immediately after falsework release.
- Some structures remain above the "before prestressing" elevation when the falsework is released.

Concrete Strength and Elastic Modulus

- Concrete strength and modulus of elasticity increase with age but at decreasing rates.
- Concrete strength and modulus of elasticity may vary considerably at any given age and cement content.

Structure Shortening

- There are large variations in the rate of total prestressed concrete superstructure shortening (shrinkage, elastic deformation and plastic deformation).
- Prestressed concrete structures shorten elastically when stressed.
- Prestressed concrete superstructure shortening rates decrease with age.
- At the end of 2 years, the superstructure had shortened approximately 0.07' per 100' of bridge length.

End Condition

- Surveys taken on this project show that abutments and bents on spread footings frequently settle after the falsework is released.
- Abutment diaphragms have been found to slide in either direction on the abutment footing.

Falsework Loading

When investigating falsework loading, load cells should be placed under all load bearing posts to determine true load distribution.

Load cell readings varied by as much as 28% between mornings and afternoons. Air temperature changes apparently have a significant effect on load cell readings.

Maximum falsework stress in falsework near the center of span usually occurs several days after the bridge deck is cast. An increase of as much as 12% in the falsework load cell readings occurred after the top deck was placed (Figure 38).

Falsework post settlement is not uniform. This causes a redistribution of loads to adjacent falsework posts.

RECOMMENDATIONS

Bridge Deflection

The 16 project structures (including one continuous 3-span structure) have a creep factor (CF) of 2.95 and a standard deviation of 38.4% when all of the recommendations are applied and projected 20,000 day deflections are compared with calculated 20,000 day deflections.

Although the recommended deflection adjustment formula give good results for the structures taken as a group there were significant variations which were not accounted for by the findings of this project. These are:

(A) Structure 50-405L projected to have a slight uplift at full term deflection rather than the fully adjusted .07' deflection calculated.

(B) Structure 50-407R projected to have a full term deflection of only .032' instead of the fully adjusted .081' deflection calculated while its twin structure projected to have a full term deflection of .133' (four times as much) with no observed reason for this significant difference.

(C) Structure 53-2373L projected to have a full term deflection of 21% greater on the right side than on the left side. Again there was no observed reason for this significant difference.

The following recommended adjustments are basically empirically determined. The formulas finally selected gave the best final adjustment deflection when all factors were taken into account. A variety of formulas for each factor were considered and the one which gave the best results was selected.

Recommendations are as follows:

1. Continue to use the current "frame system" program of the Office of Structures. This M/EI deflection calculation is very satisfactory as an initial calculation.

2. Use the best known estimate of Ec at the time of stressing. On this project the mean stressing age of soffit concrete was 40 days with a standard deviation of 17 days.

3. If the aggregate shrinkage value for the aggregate is known adjust the deflection value by the following formula.

$$\% \text{ Adj.} = (\text{Agg. Shrink.} - .048) \times 250$$

If the aggregate shrinkage values are unknown then adjust on the basis of coarse aggregate special gravity (if known) as follows.

$$\% \text{ Adj.} = (2.6 - \text{S.G.}) \times 20$$

4. Adjust the deflection value for the difference in age in days (T) between the top deck concrete and the soffit concrete.

$$\% \text{ Adj.} = 25 - [(\frac{100-T}{100})^2 \times 25]$$

5. Adjust the deflection value for the skew of the structure as follows.

$$\% \text{ Adj.} = 100 \times [\frac{1-[\cos \text{skew}]^2}{2}]$$

6. Adjust the deflection value for known concrete slumps as follows.

$$\% \text{ Adj.} = [\frac{\text{Slump(Top Slab)} + \text{Slump(Bottom Slab)}}{2} - 3.5] \times 2.0$$

7. Use a creep factor (CF) of 3.0 to determine final deflection (20,000 days - 54.7 yrs.).

8. Use a creep factor of 2.5 to determine deflection at 4.0 years.

Structure Shortening

- Use a factor of 0.07'/100' of structure length to estimate structure shortening at 2 years.

End Condition

- Consider movement of the abutment diaphragm in either longitudinal direction.

Falsework Loading

- Continue to increase the maximum design load on falsework posts by 50% over that calculated.

IMPLEMENTATION

Falsework design loads at traffic openings on California bridges have been increased by 50%. Section 51-1.06A of the January 1975 California Standard Specifications has been revised to read as follows:

> "The vertical load used for the design of falsework posts and towers which support the portion of falsework over openings, shall be increased to not less than 150 percent of the design load calculated in accordance with the provisions for design load previously specified."

REFERENCES

1. Skew Parameter Studies - An Implementation of the Finite Element Program, CELL by Mark R. Wallace, December 1975, State of California, Business and Transportation Agency, Division of Structures, Office of Research, Sacramento, California.

2. Symposium on Creep or Concrete - Neville, A. M. and Meyers, B. L., ACI Pub. SP-9, 1964.

3. Concrete Creep Study - Neal, B. F. and Mason, P. E., Caltrans Lab, FHWA-CA-TL-S148-77-08.

APPENDIX

SIMPLE SPAN CAST-IN-PLACE PRESTRESSED CONCRETE BOX GIRDER

Bridge Name	Bridge Number	Span Length	Width	Skew	Abut. Bearing Type	Aggregate Shinkage	Plan Camber
Meyer Road UC	20-242R	157.1' Ave	42'-0"	50.5° Ave	Abut. 1 Fixed Abut. 2 Hinged		0.17'
Meyer Road UC	20-242L	145.3' Ave	42'-0"	44.7° Ave	Abut. 1 Fixed Abut. 2 Hinged		0.17'
South Geyserville UC	20-243R	136'-6"	42'-0"	0°	Abut. 1 Hinged Abut. 2 Fixed		0.21'
South Geyserville UC	20-243L	136'-6"	42'-0"	0°	Abut. 1 Hinged Abut. 2 Fixed		0.21'
Canyon Road UC	20-244R	143.3'	42'-0"	4.7°	Abut. 1 Fixed Abut. 2 Hinged		0.18'
Canyon Road UC	20-244L	146.8' Ave	42'-0"	3.7° Ave	Abut. 1 Fixed Abut. 2 Hinged		0.18'
7th Ave. UC	35-261R	142.4' Ave	43'-0"	22.5° Ave	Abut. 1 Hinged Abut. 2 Fixed	0.047%	0.26'
7th Ave. UC	35-261L	127.4' Ave	43'-0"	22.5° Ave	Abut. 1 Hinged Abut. 2 Fixed	0.047%	0.26'
Larkin Valley Road UC	36-94L	156.2'	41'-0"	31.8°	Abut. 1 Hinged Abut. 2 Fixed		0.30'
South "P" Street UC	50-405R	125.2'	42'-0"	0.4°	Abut. 1 Hinged Abut. 2 Hinged	0.033%	0.16'
South "P" Street UC	50-405L	125.2'	42'-0"	0.4°	Abut. 1 Hinged Abut. 2 Hinged	0.033%	0.16'
Madison St. UC	50-407R	125.2'	42'-0"	0.4°	Abut. 1 Hinged Abut. 2 Hinged	0.033%	0.16'
Madison St. UC	50-407L	125.2'	42'-0"	0.4°	Abut. 1 Hinged Abut. 2 Hinged	0.033%	0.16'

TABLE 1

SIMPLE SPAN CAST-IN-PLACE PRESTRESSED CONCRETE BOX GIRDER

Bridge Name	Bridge Number	Span Length	Width	Skew	Abut. Bearing Type	Aggregate Shinkage	Plan Camber
Susana Road UC (Rt half)	53-2252	168.4' Ave	48'-0"	12.4° Ave	Abut. 1 Hinged Abut. 2 Fixed	0.038%	0.25'
Long Beach Blvd. UC	53-2373 (Rt.½)	195.8' Ave	85'-0"	14.2° Ave	Abut. 1 Hinged Abut. 2 Fixed	0.047%	0.42'
Long Beach Blvd. UC	53-2373 (Lt.½)	196.2' Ave	96'-0"	14.2° Ave	Abut. 1 Hinged Abut. 2 Fixed	0.047%	0.42'
Division St. UC	57-646QR	149.2'	42'-0"	14.7°	Abut. 1 Hinged Abut. 2 Fixed	0.03%	0.24'
Buckman Springs UC	57-697R	121'-2"	42'-0"	0°	Abut. 1 Hinged Abut. 2 Hinged	0.03%	0.07'
Buckman Springs UC	57-697L	124'-2"	42'-0"	4°	Abut. 1 Hinged Abut. 2 Hinged	0.03%	0.07'
San Ysidro Blvd. UC	57-776R	175.8' Ave	68'-1"	25.5° Ave	Abut. 1 Hinged Abut. 2 Fixed	0.034%	0.25'
San Ysidro Blvd. UC	57-776L	167.5'	68'-1"	17.8°	Abut. 1 Hinged Abut. 2 Fixed	0.034%	0.25'

TABLE 2

CONTINUOUS SPAN CIP PRESTRESSED CONCRETE BOX GIRDER

Bridge Name	Bridge Number	Span Number	Span Length	Width	Skew	Abut. Bearing Type	Aggregate Shinkage	Plan Camber
Montague Expressway OC	37-375	1	184'-2"	56'-0"	37°	Abut. 1 Hinged	0.047%	0.18'
Montague Expressway OC	37-375	2	184'-9"	56'-0"	37°	Abut. 3 Hinged	0.047%	0.23'
Route 58/99 Separation	50-426R	1	129'-2"	42'-0"	0°	Abut. 1 Hinged	0.026%	0.10'
Route 58/99 Separation	50-426R	2	124'-4"	42'-0"			0.026%	0.00'
Route 58/99 Separation	50-426R	3	137'-10"	42'-0"	0°	Abut. 4 Hinged	0.026%	0.15'
Route 58/99 Separation	50-426L	1	129'-2"	42'-0"	0°	Abut. 1 Hinged	0.026%	0.10'
Route 58/99 Separation	50-426L	2	124'-4"	42'-0"			0.026%	0.00'
Route 58/99 Separation	50-426L	3	137'-10"	42'-0"	0°	Abut. 4 Hinged	0.026%	0.15'
Sunrise Highway OC	57-780	1	154'-8"	42'-0"	0°	Abut. 1 Hinged	0.030%	0.11'
Sunrise Highway OC	57-780	2	156'-8"	42'-0"	0°	Abut. 3 Hinged	0.030%	0.12'
Market Street Off-Ramp	57-842	1	118.3'	27'-6"	0°	Abut. 1 Hinged	0.034%	-0.15'
Market Street Off-Ramp	57-842	2	200.0'	27'-6"	0°		0.034%	0.37'
Market Street Off-Ramp	57-842	3	118.3'	27'-6"	0°	Abut. 4 Hinged	0.034%	-0.15'

TABLE 3

BRIDGE SHORTENING

BRIDGE NO.	DATE	DISTANCE BETWEEN POINTS (Feet)								
		DECK				FOOTING				
		LEFT SIDE	Δ LENGTH	RIGHT SIDE	Δ LENGTH	LEFT SIDE	Δ LENGTH	RIGHT SIDE	Δ LENGTH	
57-697 R	5-20-74	120.848		121.024		0.000		0.000		*Before stressing*
	5-20-74	120.846	-0.002	121.012	-0.012	0.000	0.000	0.000	0.000	*After stressing*
	5-28-74	120.843	-0.005	121.000	-0.024	+0.010	+0.010	+0.015	+0.015	*After F.W. release*
	6-20-74	120.828	-0.020	120.984	-0.040	+0.010	+0.010	+0.007	+0.007	
	8-20-74	120.800	-0.048	120.950	-0.074	+0.008	+0.008	+0.013	+0.013	
	11-20-74	120.801	-0.047	120.966	-0.058					
57-697 L	5-20-74	123.498		123.705		0.000		0.000		*Before stressing*
	5-21-74	123.490	-0.008	123.694	-0.011	0.024	-0.024	-0.024	-0.024	*After stressing*
	5-21-74	123.470	-0.028	123.680	-0.025	0.015	-0.015	-0.015	-0.015	*After F.W. release*
	5-29-74	123.469	-0.029	123.672	-0.033	0.006	-0.006	-0.006	-0.006	
	6-20-74	123.440	-0.058	123.637	-0.068	0.022	-0.022	-0.021	-0.021	
	8-20-74	123.397	-0.093	123.637	-0.068	0.018	-0.018	-0.011	-0.011	
	11-20-74	123.421	-0.069	123.637	-0.068	-0.017	0.017	-0.022	-0.022	

TABLE 4

BRIDGE SHORTENING

BRIDGE NO.	DATE	DISTANCE BETWEEN POINTS (Feet)								
		DECK				FOOTING				
		LEFT SIDE	Δ LENGTH	RIGHT SIDE	Δ LENGTH	LEFT SIDE	Δ LENGTH	RIGHT SIDE	Δ LENGTH	
50-407L	5-12-75	123.770		123.130		0.750		0.750		*Before stressing*
	5-19-75	123.750	-0.020	123.110	-0.020	0.775	+0.025	0.775	+0.025	*After stressing*
	5-22-75	123.740	-0.030	123.078	-0.052	0.775	+0.025	0.775	+0.025	*After F.W. release*
	5-29-75	123.718	-0.052	123.080	-0.050					
	6-22-75	123.715	-0.055	123.080	-0.050					
	8-20-75	123.700	-0.070	123.055	-0.075					
20-243R	2-3-75	136.184		135.953		0.743		0.852		*Before stressing*
	2-3-75	136.166	-0.018	135.941	-0.012	0.724	-0.019	0.843	-0.009	*After stressing*
	2-13-75	136.127	-0.057	135.887	-0.066	0.724	-0.019	0.851	-0.001	*After F.W. release*
	2-21-75	136.119	-0.065	135.896	-0.057	0.748	+0.005	0.852	0.000	
	3-20-75	136.108	-0.076	135.882	-0.077	0.751	+0.008	0.857	+0.005	
	5-29-75	136.080	-0.104	135.858	-0.095					
	8-15-75	136.063	-0.121	135.816	-0.137					
	6-10-76	136.08	-0.104	135.84	-0.099					

TABLE 5

BRIDGE SHORTENING

BRIDGE NO.	DATE	DISTANCE BETWEEN POINTS (Feet)								
		DECK				FOOTING				
		LEFT SIDE	Δ LENGTH	RIGHT SIDE	Δ LENGTH	LEFT SIDE	Δ LENGTH	RIGHT SIDE	Δ LENGTH	
20-243L	2-3-75	136.544		136.239						*Before stressing*
	2-3-75	136.514	-0.030	136.201	-0.038	1.030		0.993		*After stressing*
	2-13-75	136.501	-0.043	136.185	-0.054	1.052	+0.022	1.011	+0.018	*After F.W. release*
	2-20-75	136.498	-0.046	136.190	-0.049	1.056	+0.026	1.018	+0.025	
	3-20-75	136.491	-0.053	136.178	-0.061	1.054	+0.024	1.018	+0.025	
	5-29-75	136.463	-0.081	136.152	-0.087					
	8-15-75	136.437	-0.107	136.112	-0.127					
	6-10-76	136.45	-0.094	136.13	-0.109					
20-244R	8-12-74	142.412		142.581		0.590		0.681		*Before stressing*
	8-12-74	142.359	-0.053	142.526	-0.055	0.561	-0.029	0.651	-0.030	*After stressing*
	8-14-74	142.339	-0.073	142.512	-0.069	0.568	-0.022	0.660	-0.021	*After F.W. release*
	8-22-74	142.339	-0.073	142.522	-0.059	0.603	+0.013	0.691	+0.010	
	9-13-74	142.334	-0.078	142.503	-0.078	0.606	+0.016	0.689	+0.008	
	11-19-74	142.321	-0.091	142.483	-0.098	0.605	+0.015			
	2-13-75	142.336	-0.076	142.482	-0.099	0.604	+0.014			
	9-5-75	142.281	-0.131	142.429	-0.152					
	6-9-76	142.33	-0.082	142.45	-0.131					

TABLE 6

BRIDGE SHORTENING

BRIDGE NO.	DATE	DISTANCE BETWEEN POINTS (Feet)								
		DECK				FOOTING				
		LEFT SIDE	Δ LENGTH	RIGHT SIDE	Δ LENGTH	LEFT SIDE	Δ LENGTH	RIGHT SIDE	Δ LENGTH	
20-244L	8-12-74	143.588		148.704		0.864		0.927		*Before stressing*
	8-12-74	143.546	-0.042	148.655	-0.049	0.837	-0.027	0.893	-0.034	*After stressing*
	8-13-74	143.523	-0.065	148.623	-0.081	0.840	-0.024	0.899	-0.028	*After F.W. release*
	8-22-74	143.524	-0.064	148.642	-0.062	0.871	+0.007	0.938	+0.011	
	9-13-74	143.526	-0.062	148.637	-0.067	0.870	+0.006	0.935	+0.008	
	11-19-74	143.490	-0.098	148.599	-0.105					
	2-13-75	143.501	-0.087	148.595	-0.109					
	9-5-75	143.446	-0.142	148.537	-0.167					
	6-11-76	143.45	-0.138	148.58	-0.124					
20-242L	10-25-74	142.135		149.317		1.345		1.805		*Before stressing*
	10-29-74	142.106	-0.029	149.289	-0.028	1.335	-0.010	1.789	-0.016	*After stressing*
	10-30-74	142.106	-0.029	149.274	-0.043	1.338	-0.007	1.790	-0.015	*After F.W. release*
	11-6-74	142.101	-0.034	149.247	-0.070	1.338	-0.007	1.795	-0.010	
	12-4-74	142.095	-0.040	149.272	-0.045	1.339	-0.006	1.798	-0.007	
	2-5-75	142.078	-0.057	149.261	-0.056	1.322	-0.023	1.760	-0.045	
	5-29-75	142.029	-0.106	149.233	-0.084					
	6-10-76	142.03	-0.105	149.20	-0.117					

TABLE 7

BRIDGE SHORTENING

BRIDGE NO.	DATE	DISTANCE BETWEEN POINTS (Feet)								
		DECK				FOOTING				
		LEFT SIDE	Δ LENGTH	RIGHT SIDE	Δ LENGTH	LEFT SIDE	Δ LENGTH	RIGHT SIDE	Δ LENGTH	
20-242R	10-25-74	148.458		173.449		1.295		1.405		*Before stressing*
	10-28-74	148.439	-0.019	173.430	-0.019	1.278	-0.017	1.378	-0.027	*After stressing*
	10-29-74	148.436	-0.022	173.421	-0.028	1.291	-0.004	1.383	-0.022	*After F.W. release*
	11-6-74	148.409	-0.049	173.382	-0.067	1.280	-0.015	1.383	-0.022	
	12-4-74	148.396	-0.062	173.371	-0.078	1.279	-0.016	1.379	-0.026	
	2-5-75			173.381	-0.068	1.275	-0.020	1.370	-0.035	
	5-24-75	148.356	-0.102	173.355	-0.094					
	6-10-76	148.34	-0.118	173.59	-0.141					
36-94L	9-23-74	152.786		152.248		0.750		0.750		*Before stressing*
	9-24-74	152.730	-0.056	152.190	-0.058	0.713	-0.037	0.713	-0.037	*After stressing*
										After F.W. release
	9-3-75	152.645	-0.141	152.110	-0.138					

TABLE 8

BRIDGE SHORTENING

BRIDGE NO.	DATE	DISTANCE BETWEEN POINTS (Feet)								
		DECK				FOOTING				
		LEFT SIDE	Δ LENGTH	RIGHT SIDE	Δ LENGTH	LEFT SIDE	Δ LENGTH	RIGHT SIDE	Δ LENGTH	
57-780	3-21-74	308.742		308.838		0.000				*Before stressing*
	3-25-74	308.645	-0.097	308.770	-0.064	0.055	-0.055			*After stressing*
	3-26-74	308.648	-0.094	308.749	-0.089	0.055	-0.055			*After F.W. release*
	4-2-74	308.665	-0.077	308.732	-0.106	0.050	-0.050			
	4-30-74	308.608	-0.134	308.705	-0.133	0.060	-0.060			
	6-26-74	308.532	-0.210	308.640	-0.198	0.036	-0.036			
	9-26-74	308.521	-0.221	308.631	-0.207					
	3-25-75	308.455	-0.287	308.600	-0.238					
	7-7-76	308.46	-0.282	308.60	-0.238					
37-375	8-2-74	365.770		360.840		3.000		1.500		*Before stressing*
	8-6-74	365.580	-0.190	360.710	-0.130	2.920	-0.080	1.420	-0.080	*After stressing*
	10-9-74	365.390	-0.380	360.570	-0.270					*After F.W. release*
	9-4-75	365.355	-0.415	360.540	-0.300					

TABLE 9

BRIDGE SHORTENING

BRIDGE NO.	DATE	DISTANCE BETWEEN POINTS (Feet)								
		DECK				FOOTING				
		LEFT SIDE	Δ LENGTH	RIGHT SIDE	Δ LENGTH	LEFT SIDE	Δ LENGTH	RIGHT SIDE	Δ LENGTH	
50-426L	1-13-75	389.190		389.845		0.000		0.000		*Before stressing*
	1-14-75	389.078	-0.112	389.720	-0.125	-0.065	-0.065	-0.065	-0.065	*After stressing*
	1-16-75	389.100	-0.090	389.734	-0.111	0.000	0.000	0.000	0.000	*After F.W. release*
	1-23-75	389.040	-0.150	389.698	-0.147					
	2-27-75	389.030	-0.160	389.690	-0.155					
	4-17-75	389.007	-0.180	389.635	-0.210					
	7-16-75	388.938	-0.25	389.580	-0.265					
50-426R	12-26-74	389.285		387.361		0.000		0.000		*Before stressing*
	12-31-74	389.233	-0.052	387.258	-0.103	0.070	-0.070	0.070	-0.070	*After stressing*
	1-3-75	389.170	-0.115	387.251	-0.110					*After F.W. release*
	1-10-75	389.200	-0.200	387.246	-0.115					
	2-3-75	389.168	-0.168	387.226	-0.135					
	4-3-75	389.040	-0.245	387.135	-0.226					
	7-8-75	389.030	-0.250	387.130	-0.231					

TABLE 10

'M' STREET UC — FALSEWORK LOADS

Date Time	FW Bent	Load Cell Readings 1	2	3	4	* Lbs. Total	** Level Reading ① (ft.)	** HI ② (ft.)	** ②-①-1' = △ (ft.)
5-31-73 0715 Initial Stem Pour	2	Ø	Ø	Ø	Ø	Ø	4.00	5.00	Ø
	3	Ø	Ø	Ø	Ø	Ø	4.00	5.00	Ø
5-31-73 1100 Pour @ B-3	2	Ø	Ø	Ø	Ø	Ø	3.895	4.895	Ø
	3	9,200	11,900	9,200	5,700	36,000	3.875	4.895	Ø
5-31-73 1345 Pour @ Midspan	2	5,500	Ø	3,500	3,500	12,500	3.82	4.83	.01
	3	12,500	17,200	13,800	8,200	51,700	3.81	4.83	.02
5-31-73 1530	2	8,800	6,900	9,500	6,500	31,700	3.69	4.71	.03
	3	14,500	19,500	15,800	9,500	59,300	3.67	4.71	.04

* 1000 lb = 453 kg
** 1 ft = .303m

TABLE 11

'M' STREET UC FALSEWORK LOADS

Date Time	FW Bent	Load Cell Readings 1	2	3	4	** Lbs. Total	*** Level Reading ① (ft.)	*** HI ② (ft.)	*** ②-①-1' = Δ (ft.)
5-31-73 Pour Complete @ 2100 hrs.	2	12,500	13,000	17,000	12,000	54,500	4.050	5.095	.045
	3	13,800	18,800	15,000	9,000	61,600	4.050	5.095	.045
6-7-73 @ 1430 hrs.	2	17,500	21,000	23,000	13,500	75,000	3.620	4.685	.065
	3	16,500	23,000	17,500	11,200	68,200	3.625	4.685	.060
6-8-73 @ 0705 hrs.	2	14,500	13,500	17,000	13,000	58,000	3.738	4.800	.062
	3	16,200	20,800	17,500	11,500	66,000	3.740	4.800	.060
6-8-73 @ 1700 hrs air temp. *105°F	2	17,600	21,200	23,500	13,500	75,800	3.897	4.960	.063
	3	17,000	23,800	17,800	11,200	69,800	3.900	4.960	.060

* °C = (°F - 32)/1.8
** 1000 lb = 453 kg
*** 1 ft = .303 m

TABLE 12

'M' STREET UC   FALSEWORK LOADS

Date Time	FW Bent	Load Cell Readings 1	2	3	4	** Lbs. Total	*** Level Reading ① (ft.)	*** HI ② (ft.)	*** ②-①-1' = Δ (ft.)
6-11-73 0720 hrs	2	13,800	13,800	15,800	13,000	56,400	3.682	4.750	.068
	3	16,000	20,300	17,200	11,200	64,700	3.688	4.750	.062
6-11-73 1500 hrs * 90°F	2	16,300	20,900	22,500	13,300	73,000	4.070	5.139	.069
	3	17,000	23,500	17,600	11,200	69,300	4.075	5.139	.064
6-12-73 0710 hrs * 65°F	2	14,500	13,800	16,500	13,300	58,100	3.830	4.900	.070
	3	16,500	21,000	17,700	11,500	66,700	3.835	4.900	.065
6-12-73 1700 hrs * 91°F	2	17,200	21,200	23,000	13,500	74,900	3.640	4.710	.070
	3	17,200	24,000	17,800	11,700	70,700	3.642	4.710	.068

* °C = (°F - 32)/1.8
** 1000 lb = 453 kg
*** 1 ft = .303 m

TABLE 13

'M' STREET UC FALSEWORK LOADS

Date Time	FW Bent	Load Cell Readings 1	2	3	4	** Lbs. Total	*** Level Reading ① (ft.)	*** HI ② (ft.)	*** ②-①-1' = Δ (ft.)
6-13-73 0715 hrs. * 64°F clear	2	14,500	13,800	17,000	13,300	58,600	3.610	4.680	.070
	3	16,300	21,000	17,700	11,700	66,700	3.615	4.680	.065
6-13-73 1610 hrs. * 76°F overcast	2	15,800	19,200	19,500	12,800	67,300	3.700	4.770	.070
	3	16,500	21,700	17,400	11,200	66,800	3.705	4.770	.065
6-14-73 0705 hrs. * 55°F hazy sky	2	14,500	14,000	17,500	13,600	59,600	3.718	4.790	.072
	3	16,800	21,700	18,200	11,800	68,500	3.722	4.790	.068
6-14-73 1615 hrs. *73°F clear	2	16,800	21,500	22,800	13,600	74,700	3.607	4.680	.073
	3	17,500	24,000	18,300	12,000	71,800	3.612	4.680	.068

* °C = (°F - 32)/1.8
** 1000 lb = 453 kg
*** 1 ft = .303 m

TABLE 14

'M' STREET UC   FALSEWORK LOADS

Date Time	FW Bent	Load Cell Readings				** Lbs. Total	*** Level Reading ① (ft.)	*** HI ② (ft.)	*** ②-①-1' = Δ (ft.)
		1	2	3	4				
6-15-73 0715 hrs. *58°F clear	2	15,000	14,500	17,500	13,800	60,800	3.698	4.770	.072
	3	17,300	22,000	18,500	12,000	69,800	3.702	4.770	.068
6-15-73 1600 hrs. *79°F clear warm	2	17,200	21,300	22,800	13,600	74,900	4.270	5.340	.070
	3	17,900	24,100	18,400	11,800	72,200	4.273	5.340	.067
6-18-73 0800 hrs *62°F clear	2	15,200	15,200	19,000	13,700	63,100	3.935	5.009	.074
	3	17,500	22,500	18,300	11,800	70,100	3.943	5.009	.066
6-18-73 1645 hrs. *87°F warm	2	17,200	21,000	22,200	13,300	73,700	3.342	4.410	.068
	3	17,800	24,000	18,200	11,700	71,700	3.335	4.410	.075

* °C = (°F - 32)/1.8
** 1000 lb = 453 kg
*** 1 ft = .303 m

TABLE 15

'M' STREET UC FALSEWORK LOADS

Date Time	FW Bent	Load Cell Readings 1	2	3	4	** Lbs. Total	*** Level Reading ① (ft.)	*** HI ② (ft.)	*** ②-①-1' = Δ (ft.)
6-19-73 0720 hrs. *67°F	2	15,000	14,300	18,200	13,300	60,800	3.385	4.460	.075
	3	17,000	21,600	18,000	11,700	68,300	3.392	4.460	.068
6-19-73 1610 hrs. *99°F	2	17,500	21,000	22,200	13,300	74,000	3.277	4.350	.073
	3	17,800	23,700	18,000	11,800	71,300	3.282	4.350	.068
6-20-73 0715 hrs. *72°F	2	14,800	14,000	17,200	13,000	59,000	3.350	4.420	.070
	3	17,000	21,000	17,700	11,200	66,900	3.352	4.420	.068
6-20-73 1700 hrs. *102°F	2	17,300	21,000	21,500	13,300	73,100	3.387	4.460	.073
	3	17,500	23,300	17,800	11,300	69,900	3.393	4.460	.067

* $^{\circ}C = (^{\circ}F - 32)/1.8$

** 1000 lb = 453 kg

*** 1 ft = .303 m

TABLE 16

'M' STREET UC   FALSEWORK LOADS

Date Time	FW Bent	Load Cell Readings 1	2	3	4	** Lbs. Total	*** Level Reading ① (ft.)	*** HI ② (ft.)	*** ②-①-1' = Δ (ft.)
6-21-73 0715 hrs. * 80°F	2	14,800	14,200	17,300	13,000	59,300	3.327	4.400	.073
	3	16,500	20,800	17,500	11,200	66,000	3.332	4.400	.068
6-21-73 1540 hrs *108°F	2	12,200	21,000	21,200	13,300	67,700	3.328	4.400	.072
	3	17,300	23,000	17,500	11,200	69,000	3.332	4.400	.068
6-22-73 0800 hrs * 80°F	2	14,300	12,900	16,500	12,900	56,600	4.006	5.077	.071
	3	16,700	20,100	17,000	11,000	64,800	4.010	5.077	.067
6-22-73 1545 hrs. * 93°F	2	16,500	20,400	21,500	13,400	71,800	3.512	4.592	.080
	3	17,200	22,600	17,200	11,100	68,100	3.520	4.592	.072

* °C = (°F - 32)/1.8
** 1.000 lb = 453 kg
*** 1 ft = .303 m

TABLE 17

'M' STREET UC FALSEWORK LOADS

Date Time	FW Bent	Load Cell Readings 1	Load Cell Readings 2	Load Cell Readings 3	Load Cell Readings 4	** Lbs. Total	*** Level Reading ① (ft.)	*** HI ② (ft.)	*** ②-①-1' = Δ (ft.)
6-25-73 0720 hrs. * 74°F	2	14,200	13,200	17,200	13,000	57,600	3.422	4.500	.078
	3	16,500	20,700	17,200	11,300	65,700	3.428	4.500	.072
6-25-73 1715 hrs. * 97°F	2	17,200	21,000	21,800	13,500	73,500	3.262	4.340	.078
	3	17,400	22,800	17,000	11,200	68,400	3.270	4.340	.070
6-26-73 0715 hrs. * 80°F	2	15,000	13,800	17,400	13,000	59,200	3.402	4.480	.078
	3	16,800	20,800	17,000	11,200	65,800	3.408	4.480	.072
6-26-73 1630 hrs. * 104°F	2	17,200	20,400	21,000	13,300	71,900	3.324	4.400	.076
	3	17,200	22,300	16,800	11,000	67,300	3.330	4.400	.070

* °C = (°F - 32)/1.8
** 1000 lb = 453 kg
*** 1 ft = .303 m

TABLE 18

'M' STREET UC FALSEWORK LOADS

Date Time	FW Bent	Load Cell Readings 1	2	3	4	** Lbs. Total	*** Level Reading ①(ft.)	*** HI ②(ft.)	*** ②-①-1' = △ (ft.)
6-27-73 0715 hrs. * 86°F	2	15,000	14,000	17,200	13,000	59,200	3.293	4.370	.077
	3	16,800	20,800	16,800	11,000	65,400	3.300	4.370	.070
6-27-73 1630 hrs. * 107°F	2	17,000	20,300	20,800	13,200	71,300	3.303	4.380	.077
	3	16,800	21,800	16,500	11,000	66,100	3.310	4.380	.070
6-28-73 0730 hrs. * 80°F	2	13,800	12,200	15,700	12,500	54,200	3.353	4.430	.077
	3	16,000	19,800	16,200	10,800	62,800	3.360	4.430	.070
6-28-73 1530 hrs. * 103°F	2	16,500	11,500	21,500	13,300	62,800		—	—
	3	17,000	22,000	16,800	11,000	66,800	—	—	—

Note (with arrow pointing to 11,500, load cell 2): Ice pack applied to this load cell @ 1130 hrs. and left in place. Load cell reading @ 1130 prior to applied ice: 17,000**

* °C = (°F - 32)/1.8
** 1000 lb = 453 kg
*** 1 ft = .303 m

TABLE 19

'M' STREET UC FALSEWORK LOADS

Date Time	FW Bent	Load Cell Readings				** Lbs. Total	*** Level Reading ① (ft.)	*** HI ② (ft.)	*** ②-①-1' = Δ (ft.)
		1	2	3	4				
6-29-73 0720 hrs. * 76°F	2	12,800	12,000	16,000	12,700	53,500	—	—	—
	3	16,000	19,500	16,200	10,800	62,500	—	—	—
7-19-73 1600 hrs. *86°F	2	16,800	21,000	21,000	14,000	72,800	3.555	4.640	.085
	3	18,200	22,800	18,000	11,800	70,800	3.560	4.640	.080
7-20-73 1700 hrs. * 78°F Deck pour complete	2	21,500	23,500	26,000	17,500	88,500	3.655	4.750	.095
	3	16,200	24,500	30,000	23,500	94,200	3.665	4.750	.085
7-25-73 0940 hrs. *81°F	2	22,500	23,500	26,500	18,200	90,700	No Ice		
	3	28,000	35,500	31,000	20,000	114,500			

* °C = (°F - 32)/1.8
** 1000 lb = 453 kg
*** 1 ft = .303 m

TABLE 20

'M' STREET UC FALSEWORK LOADS

Date Time	FW Bent	Load Cell Readings 1	2	3	4	** Lbs. Total	*** Level Reading ① (ft.)	*** HI ② (ft.)	*** ②-①-1' = Δ (ft.)
7-25-73	2	Icing @1045	of cells hrs. 25	completed # ice/cell					
	3	Icing @1130	of cells hrs, 25	completed # ice/cell					
7-25-73 1320 hrs. *91°F	2	19,000	19,200	20,500	16,000	74,700			
	3	25,000	31,000	28,000	18,000	102,000			
7-25-73 1530 hrs. *97°F	2	20,000	20,000	21,500	16,000	77,500	3.614	4.730	0.116
	3	24,000	31,000	28,000	18,500	101,500	3.627	4.730	0.103
No Ice 7-26-73 1625 hrs. *104°F	2	23,500	26,000	20,500	20,000	90,000			
	3	24,800	32,200	23,500	19,000	99,500			

* °C = (°F - 32)/1.8
** 1000 lb = 453 kg
*** 1 ft = .303 m

TABLE 21

'M' STREET UC FALSEWORK LOADS

Date Time	FW Bent	Load Cell Readings 1	2	3	4	** Lbs. Total	*** Level Reading ①(ft.)	*** HI ②(ft.)	*** ②-①-1' = △ (ft.)
7-27-73 0710 hrs. *74°F	2	21,000	18,500	20,000	17,800	77,300			
	3	23,200	28,500	23,800	19,000	94,500			
7-31-73 0715 hrs. *74°F	2	21,000	19,500	19,500	16,800	76,800	3.535	4.660	0.125
	3	22,500	26,000	23,300	18,500	90,300	3.545	4.660	0.115
7-31-73 1550 hrs. *100°F	2	22,500	25,800	23,000	17,500	88,700	4.205	5.330	0.125
	3	25,500	32,000	23,000	18,000	98,500	4.216	5.330	0.114
8-7-73 0600 hrs. prior to stressing	2	19,750	19,500	17,750	16,000	73,000			
	3	22,000	26,000	23,500	18,000	89,500			

* °C = (°F - 32)/1.8
** 1000 lb = 453 kg
*** 1 ft = .303 m

TABLE 22

'M' STREET UC   FALSEWORK LOADS

Date Time	FW Bent	Load Cell Readings				** Lbs. Total	Level Reading ① (ft.)	HI ② (ft.)	②-①-1' = Δ (ft.)
		1	2	3	4				
8-7-73 after stressing	2	13,200	16,800	14,500	9,000	53,500			
	3	16,500	19,000	13,000	10,000	58,500			

** 1000 lb = 453 kg

TABLE 23

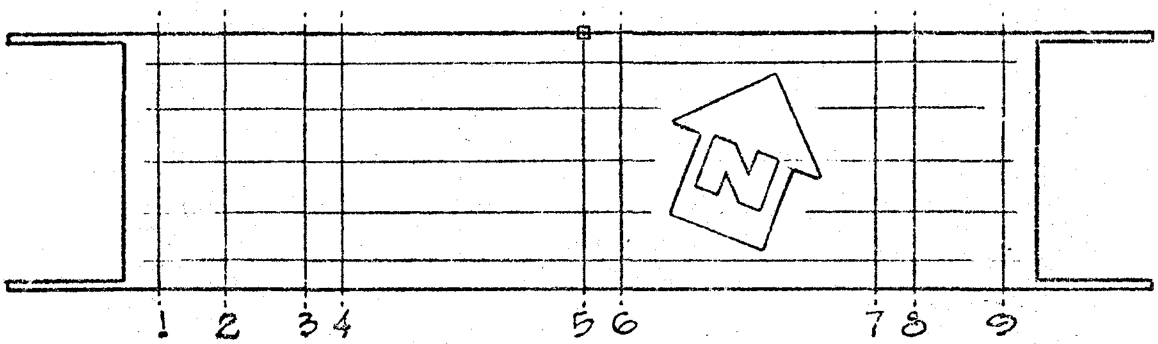
OLD OREGON TRAIL FALSEWORK LOAD CELL STUDY

Cell No.	*** Total Settlement	LOAD IN KIPS** Falsework + Cell	Calc.	Bot. Slab + Stem Cell	Calc.	+ Top Slab (Total) Cell	Calc.	After Prestress Cell
2	0	9.0	4.5	18.0	6.3	28.5	15.4	11.0
5	0	1.0	3.6	8.0	13.7	16.0	19.5	3.0
* 24	0.01'	2.0	3.6	9.5	13.7	17.0	19.5	2.5
12	0.015'	10.0	3.6	34.0	17.3	55.0	27.7	20.0
* 4	0.02'	3.0	3.6	12.0	17.3	21.0	27.7	6.0
9	0.03'	4.5	3.6	18.5	17.3	29.0	27.7	12.0
* 23	0.025'	2.0	3.6	12.5	17.3	21.0	27.7	8.0
6	0.02'	2.5	3.6	8.0	13.7	12.5	19.5	5.0
* 7	0	7.0	3.6	16.5	13.7	23.5	19.5	12.5
3	0.015'	3.5	4.5	6.0	6.3	10.5	15.4	5.0
10	0	2.0	2.5	14.5	14.3	23.5	23.1	5.0
21	0.015'	2.5	2.5	15.0	13.9	23.0	23.7	6.5
Σ =						280.5	266.4	

* Indicates double posts
** 1000 lb = 453 kg
*** 1 ft = .303 m

TABLE 24

FALSEWORK LOAD STUDY
OLD OREGON TRAIL U.C. (LT. BR.) #6-178L
LOAD CELL # 2   FALSEWORK BENT # 5



DATE	TIME	TEMPERATURE *AIR°F	*CONC°F	*** ELEV FT	POST LOAD**	REMARKS
8-29-74	1515			0.925	9.0k	PRE-POUR
8-30	1430	95	112	0.920	18.0	POUR SOFFIT & STEM
8-31	0800	77	98	0.915	18.	
9-1-74	0930	79	97	0.92	19.	
9-3	0930	76	86	0.925	19.	
9-4	1130	92			18.5	
9-9	0900	80		0.92	17.5	
9-11	1400	90			18.5	
9-16	1430			0.92	18.5	
9-17	1300	95	90	0.92	27.0	POUR DECK
9-18	0930	82	90	0.92	28.0	
9-19	0845	72	85	0.92	28.0	
9-20	0845	80	70		29.5	
9-21	1200	91			30.0	
9-25	0945	85	66		30.0	
9-27	0815	70	70		25.5	REMOVED EXT. GIR. FORMS
9-30-74	0830			0.92	11.0	PRESTRESSED

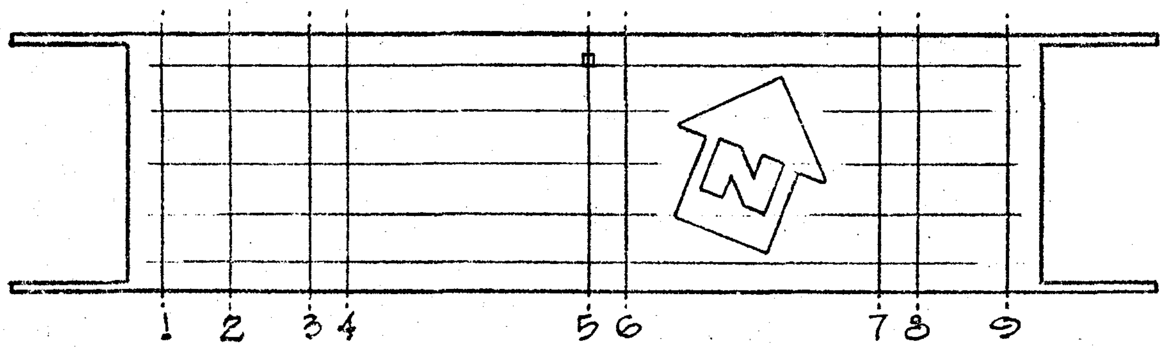
* °C = (°F - 32)/1.8
** 1000 lb = 453 kg
*** 1 ft = .303 m

TABLE 25

FALSEWORK LOAD STUDY

OLD OREGON TRAIL U.C. (LT. BR.) #6-178L

LOAD CELL # 5 FALSEWORK BENT # 5



DATE	TIME	TEMPERATURE *AIR-°F	*CONC-°F	ELEV FT	POST LOAD**	REMARKS
8-29-74	1515				1.0K	PRE-POUR
8-30	1430	95	112		8.0	POUR SOFFIT & STEM
8-31	0800	77	98		8.0	
9-1-74	0930	79	97		8.0	
9-3	0930	96	86		8.0	
9-4	1130	92			7.5	
9-9	0900	80			7.0	
9-11	1400	90			8.0	
9-16	1430				8.0	
9-17	1300	95	90		15.0	POUR DECK
9-18	0930	82	90		15.0	
9-19	0845	72	85		15.0	
9-20	0845	80	70		15.0	
9-21	1200	91			16.0	
9-25	0945	85	66		17.0	
9-27	0815	70	70		18.0	REMOVED EXT. GIR. FORMS
9-30-74	0830				3.0	PRESTRESSED

* °C = (°F - 32)/1.8

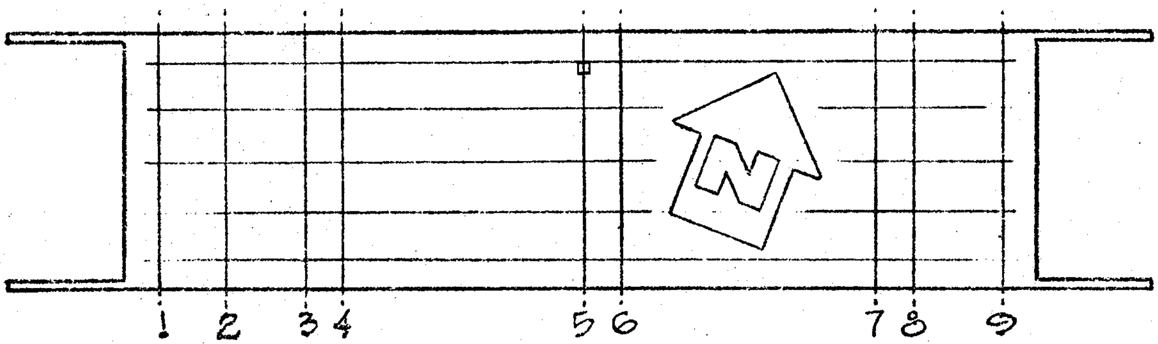
** 1000 lb = 453 kg

TABLE 26

FALSEWORK LOAD STUDY

OLD OREGON TRAIL U.C. (LT. BR.) #6-178L

LOAD CELL # 24 FALSEWORK BENT # 5



DATE	TIME	TEMPERATURE * AIR-°F	*CONC-°F	*** ELEV FT	POST LOAD**	REMARKS
8-29-74	1515			0.950	2.0K	PRE-POUR
8-30	1430	95	112	0.940	9.0	POUR SOFFIT & STEM
8-31	0800	77	98	0.940	9.5	
9-1-74	0930	79	97	0.945	10.0	
9-3	0930	76	86	0.945	10.0	
9-4	1130	92			8.5	
9-9	0900	80		0.940	9.0	
9-11	1400	90			9.5	
9-16	1430			0.94	9.5	
9-17	1300	95	90	0.935	17.0	POUR DECK
9-18	0930	82	90	0.94	17.0	
9-19	0845	72	85	0.935	17.0	
9-20	0845	80	70		16.5	
9-21	1200	91			15.5	
9-25	0945	85	66		13.0	
9-27	0815	70	70		13.0	REMOVED EXT. GIR. FORMS
9-30-74	0830			0.94	2.5	PRESTRESSED

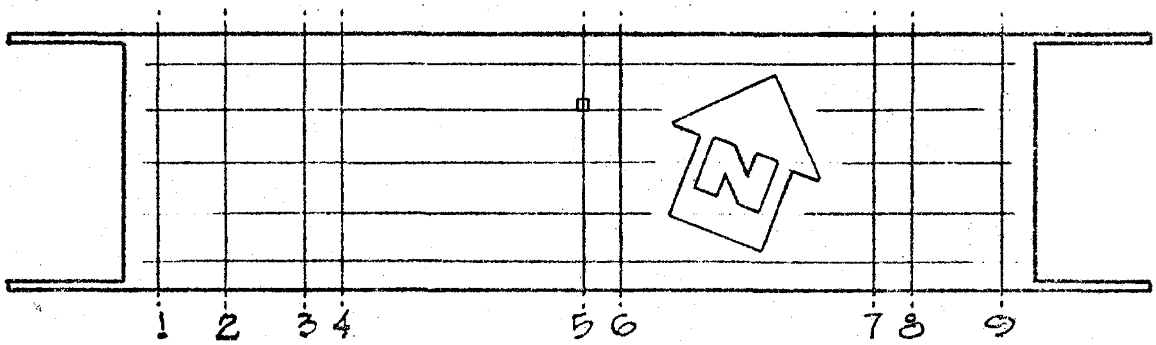
* °C = (°F - 32)/1.8
** 1000 lb = 453 kg
*** 1 ft = .303 m

TABLE 27

FALSEWORK LOAD STUDY

OLD OREGON TRAIL U.C. (LT. BR.) #G-178L

LOAD CELL # 12 FALSEWORK BENT # 5



DATE	TIME	TEMPERATURE * AIR-°F	*CONC-°F	*** ELEV FT	POST LOAD**	REMARKS
8-29-74	1515			0.850	10.0K	PRE-POUR
8-30	1430	95	112	0.840	33.0	POUR SOFFIT & STEM
8-31	0800	77	98	0.840	33.0	
9-1-74	0930	79	97	0.845	34.5	
9-3	0930	76	86	0.840	35.0	
9-4	1130	92			34.5	
9-9	0900	80		0.84	35.0	
9-11	1400	90			35.0	
9-16	1430			0.84	35.0	
9-17	1300	95	90	0.84	50.5	POUR DECK
9-18	0930	82	90	0.84	53.0	
9-19	0845	72	85	0.835	55.0	
9-20	0845	80	70		56.0	
9-21	1200	91			55.0	
9-25	0945	85	66		55.0	
9-27	0815	70	70		57.0	REMOVED EXT. GIR. FORMS
9-30-74	0830			0.835	20.0	PRESTRESSED

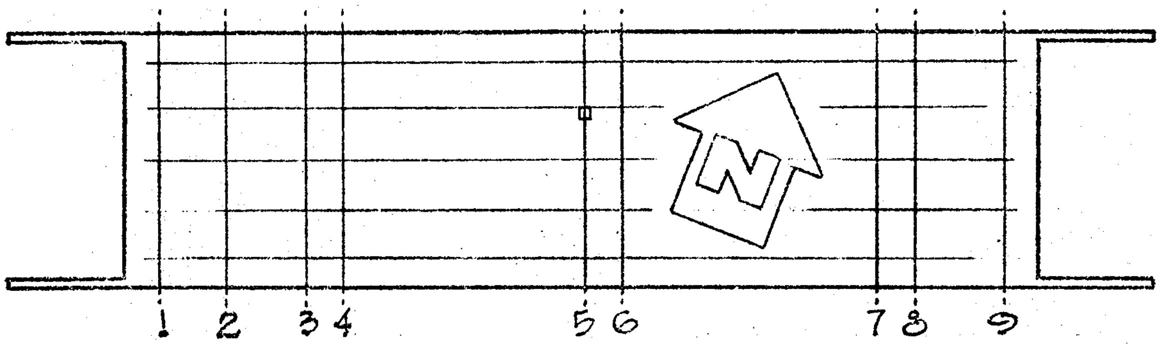
* $^{o}C = (^{o}F - 32)/1.8$
** 1000 lb = 453 kg
*** 1 ft = .303 m

TABLE 28

FALSEWORK LOAD STUDY

OLD OREGON TRAIL U.C. (LT. BR.) #6-178L

LOAD CELL # 4 FALSEWORK BENT # 5



DATE	TIME	TEMPERATURE *AIR-°F	*CONC-°F	*** ELEV FT	POST LOAD**	REMARKS
8-29-74	1515			0.850	3.0K	PRE-POUR
8-30	1430	95	112	0.840	11.0	POUR SOFFIT & STEM
8-31	0800	77	98	0.835	11.0	
9-1-74	0930	79	97	0.835	12.0	
9-3	0930	76	86	0.835	12.0	
9-4	1130	92			12.5	
9-9	0900	80		0.835	12.0	
9-11	1400	90			12.5	
9-16	1430			0.835	13.0	
9-17	1300	95	90	0.83	19.0	POUR DECK
9-18	0930	82	90	0.83	20.5	
9-19	0845	72	85	0.83	20.5	
9-20	0845	80	76		21.5	
9-21	1200	91			21.5	
9-25	0945	85	66		21.5	
9-27	0815	70	70		22.5	REMOVED EXT. GIR. FORMS
9-30-74	0830			0.83	6.0	PRESTRESSED

* °C = (°F - 32)/1.8

** 1000 lb = 453 kg

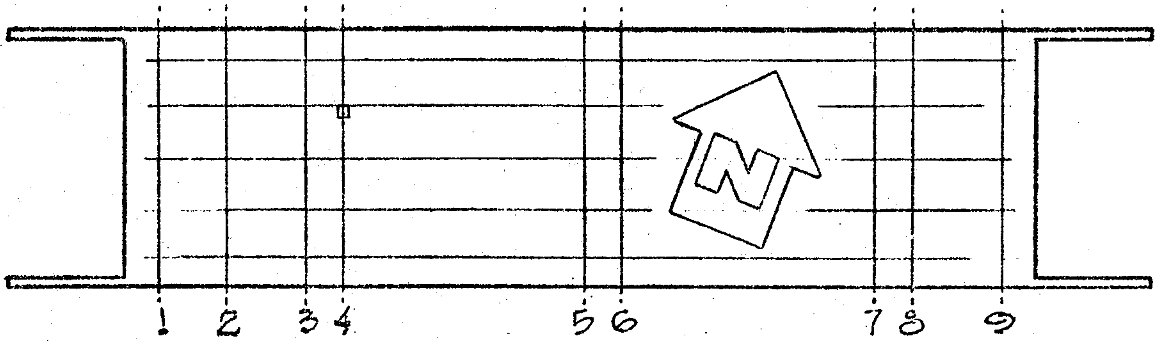
*** 1 ft = .303 m

TABLE 29

FALSEWORK LOAD STUDY

OLD OREGON TRAIL U.C. (LT. BR.) #6-178L

LOAD CELL # 9 FALSEWORK BENT # 4



DATE	TIME	TEMPERATURE * AIR-°F	*CONC°F	*** ELEV FT	POST LOAD**	REMARKS
8-29-74	1515			0.260	4.5K	PRE-POUR
8-30	1430	95	112	0.245	17.0	POUR SOFFIT & STEM
8-31	0800	77	98	0.240	18.5	
9-1-74	0930	79	97	0.245	19.0	
9-3	0930	76	86	0.240	19.5	
9-4	1130	92			18.5	
9-9	0900	80		0.24	19.5	
9-11	1400	90			19.0	
9-16	1430			0.24	19.5	
9-17	1300	95	90	0.24	27.0	POUR DECK
9-18	0930	82	90	0.235	29.0	
9-19	0845	72	85	0.235	29.5	
9-20	0845	80	70		29.5	
9-21	1200	91			29.0	
9-25	0945	85	66		29.0	
9-27	0815	70	76		29.0	REMOVED EXT. GIR. FORMS
9-30-74	0830			0.23	12.0	PRESTRESSED

* °C = (°F - 32)/1.8

** 1000 lb = 453 kg

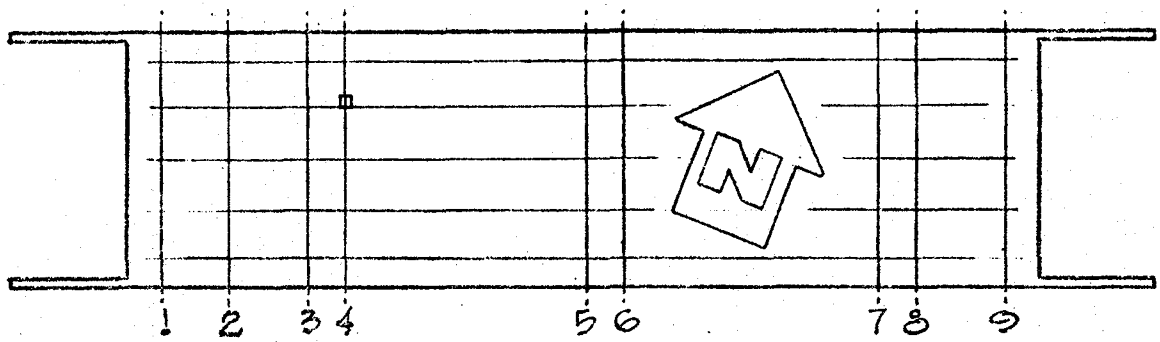
*** 1 ft = .303 m

TABLE 30

FALSEWORK LOAD STUDY

OLD OREGON TRAIL U.C. (LT. BR.) #6-173L

LOAD CELL # 23 FALSEWORK BENT # 4



DATE	TIME	TEMPERATURE * AIR °F	TEMPERATURE * CONC °F	*** ELEV FT	POST LOAD**	REMARKS
8-29-74	1515			0.125	2.0K	PRE-POUR
8-30	1430	95	112	0.110	11.5	POUR SOFFIT & STEM
8-31	0800	77	98	0.110	12.5	
9-1-74	0930	79	97	0.115	13.0	
9-3	0930	76	86	0.115	13.5	
9-4	1130	92			13.0	
9-9	0900	80		0.115	13.5	
9-11	1400	90			13.5	
9-16	1430			0.113	13.0	
9-17	1300	95	90	0.11	19.0	POUR DECK
9-18	0930	82	90	0.11	21.0	
9-19	0845	72	85	0.10	22.0	
9-20	0845	80	70		22.5	
9-21	1200	91			21.0	
9-25	0945	85	66		21.0	
9-27	0815	70	70		22.0	REMOVED EXT. GIR. FORMS
9-30-74	0830			0.10	8.0	PRESTRESSED

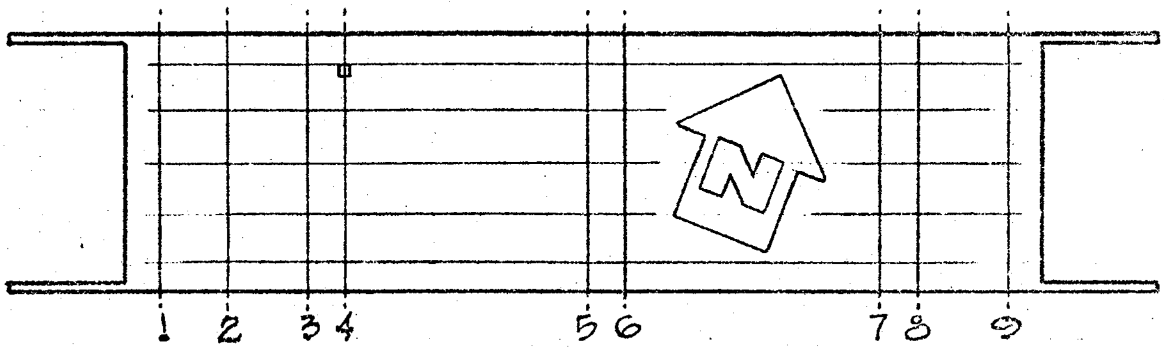
* °C = (°F - 32)/1.8
** 1000 lb = 453 kg
*** 1 ft = .303 m

TABLE 31

FALSEWORK LOAD STUDY

OLD OREGON TRAIL U.C. (LT. BR.) #G-178L

LOAD CELL #6 FALSEWORK BENT #4



DATE	TIME	TEMPERATURE *AIR-°F	*CONC°F	*** ELEV FT	POST LOAD**	REMARKS
8-29-74	1515			0.080	2.5K	PRE-POUR
8-30	1430	95	112	0.070	8.0	POUR SOFFIT & STEM
8-31	0800	77	98	0.07	8.5	
9-1-74	0930	79	97	0.070	9.0	
9-3	0930	76	86	0.070	9.0	
9-4	1130	92			8.0	
9-9	0900	80		0.070	8.0	
9-11	1400	90			8.0	
9-16	1430			0.070	8.0	
9-17	1300	95	90	0.060	12.0	POUR DECK
9-18	0930	82	90	0.06	12.5	
9-19	0845	72	85	0.06	13.0	
9-20	0845	80	70		13.0	
9-21	1200	91			12.5	
9-25	0945	85	66		12.5	
9-27	0815	70	70		13.0	REMOVED EXT. GIR. FORMS
9-30-74	0830			0.06	5.0	PRESTRESSED

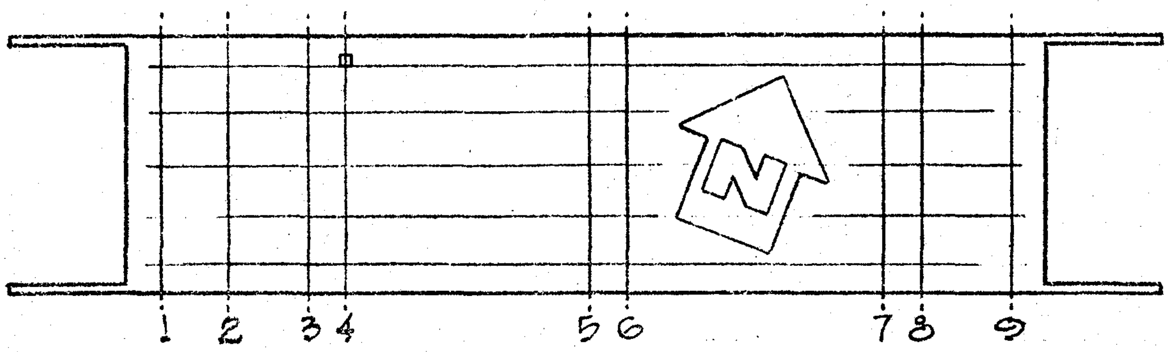
* °C = (°F - 32)/1.8
** 1000 lb = 453 kg
*** 1 ft = .303 m

TABLE 32

FALSEWORK LOAD STUDY

OLD OREGON TRAIL U.C. (LT. BR.) #6-178L

LOAD CELL # 7 FALSEWORK BENT # 4



DATE	TIME	TEMPERATURE *AIR-°F	*CONC-°F	***ELEV FT	POST LOAD**	REMARKS
8-29-74	1515				7.0K	PRE-POUR
8-30	1430	95	112		15.0	POUR SOFFIT & STEM
8-31	0800	77	98		16.0	
9-1-74	0930	79	97		17.0	
9-3	0930	76	86		17.5	
9-4	1130	92			17.0	
9-9	0900	80			17.0	
9-11	1400	90			17.5	
9-16	1430				17.0	
9-17	1300	95	90		23.0	POUR DECK
9-18	0930	82	90		23.5	
9-19	0845	72	85		23.5	
9-20	0845	80	70		24.5	
9-21	1200	91			23.5	
9-25	0945	85	66		23.5	
9-27	0815	70	70		23.0	REMOVED EXT. GIR. FORMS
9-30-74	0830				12.5	PRESTRESSED

* $^{o}C = (^{o}F - 32)/1.8$

** 1000 lb = 453 kg

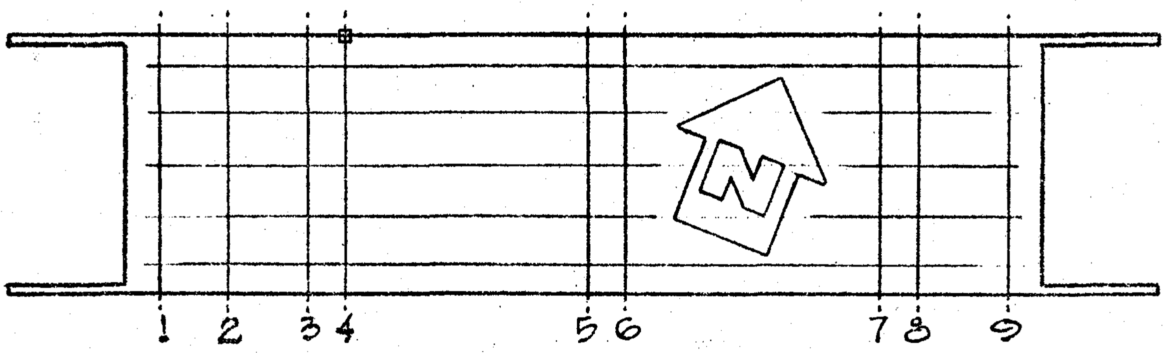
*** 1 ft = .303 m

TABLE 33

FALSEWORK LOAD STUDY

OLD OREGON TRAIL U.C. (LT. BR.) #6-178L

LOAD CELL # 3 FALSEWORK BENT # 4



DATE	TIME	TEMPERATURE *AIR-°F	*CONC-°F	*** ELEV FT	POST LOAD**	REMARKS
8-29-74	1515			0.175	3.5K	PRE-POUR
8-30	1430	95	112	0.170	6.0	POUR SOFFIT & STEM
8-31	0800	77	98	0.170	6.0	
9-1-74	0930	79	97	0.165	6.0	
9-3	0930	76	86	0.170	6.0	
9-4	1130	92			6.5	
9-9	0900	80		0.170	6.0	
9-11	1400	90			6.5	
9-16	1430			0.170	7.0	
9-17	1300	95	90	0.165	10.0	POUR DECK
9-18	0930	82	90	0.165	10.5	
9-19	0845	72	85	0.165	11.0	
9-20	0845	80	70		11.0	
9-21	1200	91			11.0	
9-25	0945	85	66		11.0	
9-27	0815	70	70		7.5	REMOVED EXT. GIR. FORMS
9-30-74	0830			0.16	5.0	PRESTRESSED

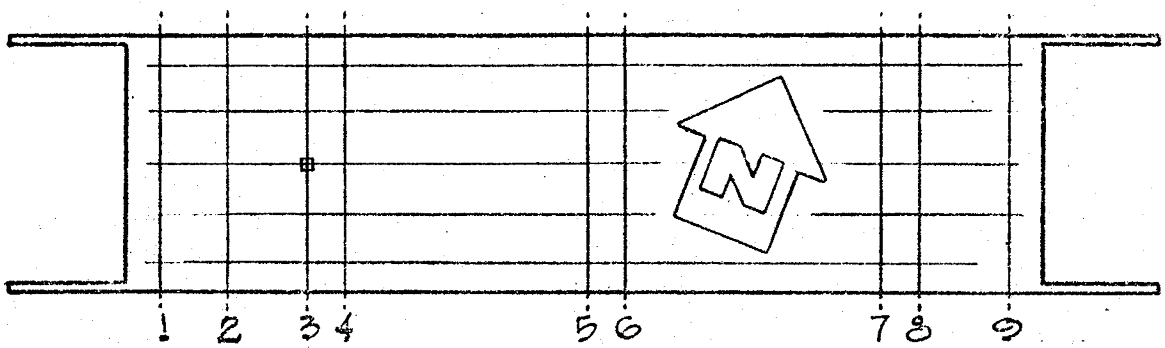
* °C = (°F - 32)/1.8
** 1000 lb = 453 kg
*** 1 ft = .303 m

TABLE 34

FALSEWORK LOAD STUDY

OLD OREGON TRAIL U.C. (LT. BR.) #6-178L

LOAD CELL # 10 FALSEWORK BENT # 3



DATE	TIME	TEMPERATURE * AIR-°F	*CONC°F	*** ELEV FT	POST LOAD**	REMARKS
8-29-74	1515			8.89	2.0K	PRE-POUR
8-30	1430	95	112	8.88	15.0	POUR SOFFIT & STEM
8-31	0800	77	98	8.88	14.0	
9-1-74	0930	79	97	8.88	14.5	
9-3	0930	76	86	8.88	14.0	
9-4	1130	92			15.0	
9-9	0900	80		8.88	14.0	
9-11	1400	90			14.0	
9-16	1430			8.88	14.0	
9-17	1300	95	90	8.88	20.5	POUR DECK
9-18	0930	82	90	8.88	22.5	
9-19	0845	72	85	8.88	24.0	
9-20	0845	80	70		24.0	
9-21	1200	91			23.0	
9-25	0945	85	66		23.0	
9-27	0815	70	70		23.5	REMOVED EXT. GIR. FORMS
9-30-74	0830			8.89	5.0	PRESTRESSED

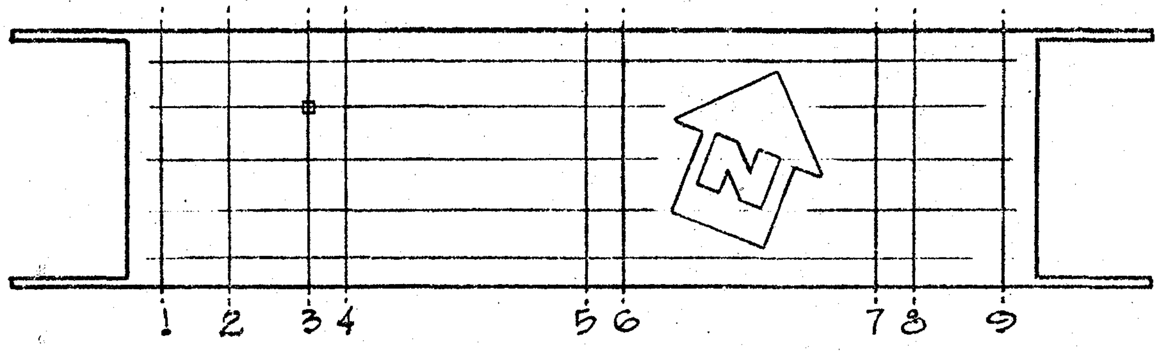
* °C = (°F - 32)/1.8

** 1000 lb = 453 kg

*** 1 ft = .303 m

TABLE 35

FALSEWORK LOAD STUDY
OLD OREGON TRAIL U.C. (LT. BR.) #6-178L
LOAD CELL # 21 FALSEWORK BENT # 3



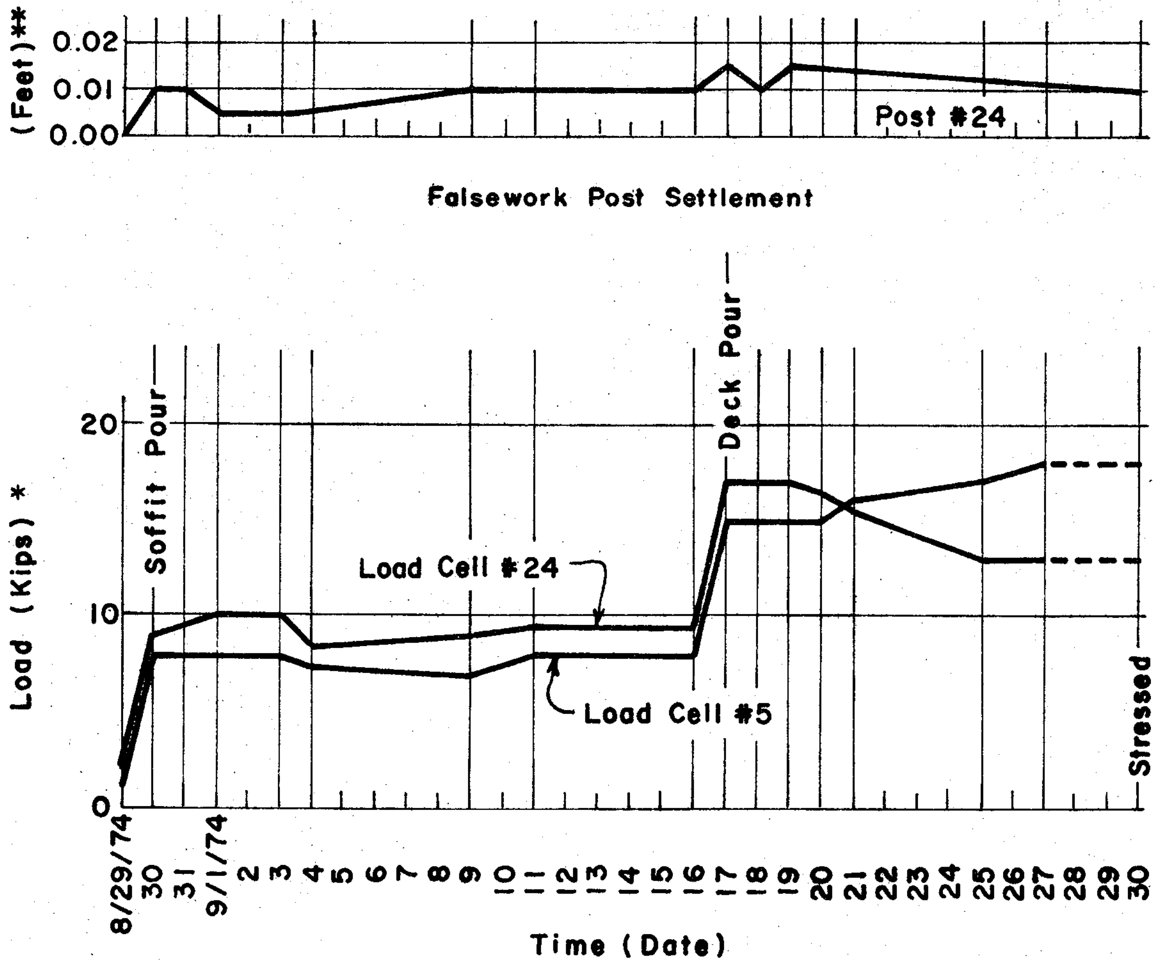
DATE	TIME	TEMPERATURE *AIR-°F	*CONC°F	*** ELEV FT	POST LOAD**	REMARKS
8-29-74	1515			8.88	2.5K	PRE-POUR
8-30	1430	95	112	8.87	15.5	POUR SOFFIT & STEM
8-31	0800	77	98	8.87	15.0	
9-1-74	0930	79	97	8.87	15.0	
9-3	0930	76	86	8.87	15.0	
9-4	1130	92			15.0	
9-9	0900	80		8.87	15.0	
9-11	1400	90			15.0	
9-16	1430			8.87	15.0	
9-17	1300	95	90	8.87	21.0	POUR DECK
9-18	0930	82	90	8.87	22.0	
9-19	0845	72	85	8.87	23.5	
9-20	0845	80	70		24.0	
9-21	1200	91			23.0	
9-25	0945	85	66		22.5	
9-27	0815	70	70		23.0	REMOVED EXT. GIR. FORMS
9-30-74	0830			8.87	6.5	PRESTRESSED

* $^{o}C = (^{o}F - 32)/1.8$

TABLE 36

** 1000 lb = 453 kg

*** 1 ft = .303 m

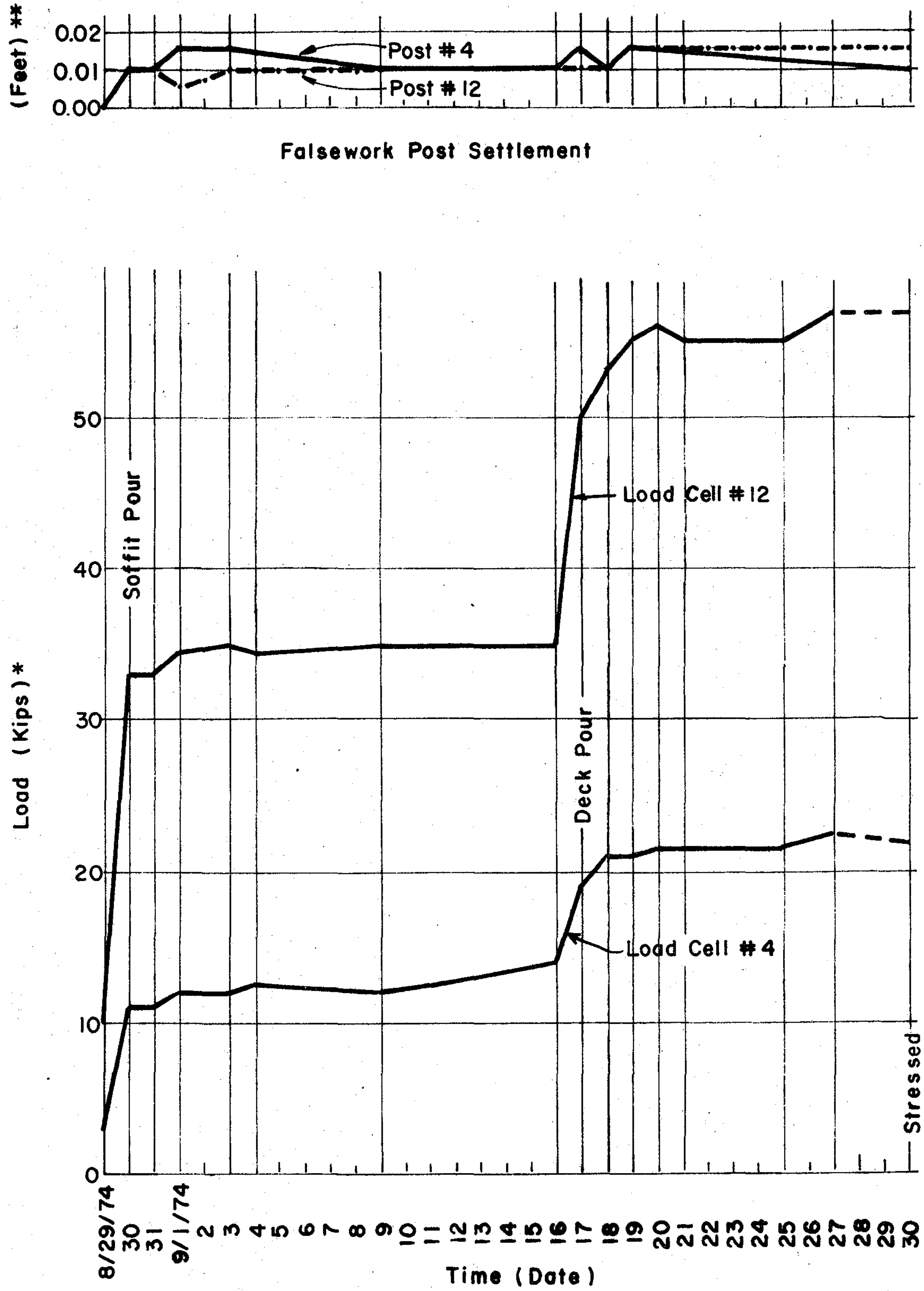


FALSEWORK LOAD CHANGES

* 1000 lb = 453 kg
** 1 ft = .303 m

Figure 37

OLD OREGON TRAIL UNDERCROSSING LEFT BRIDGE Br. No. 6-178L



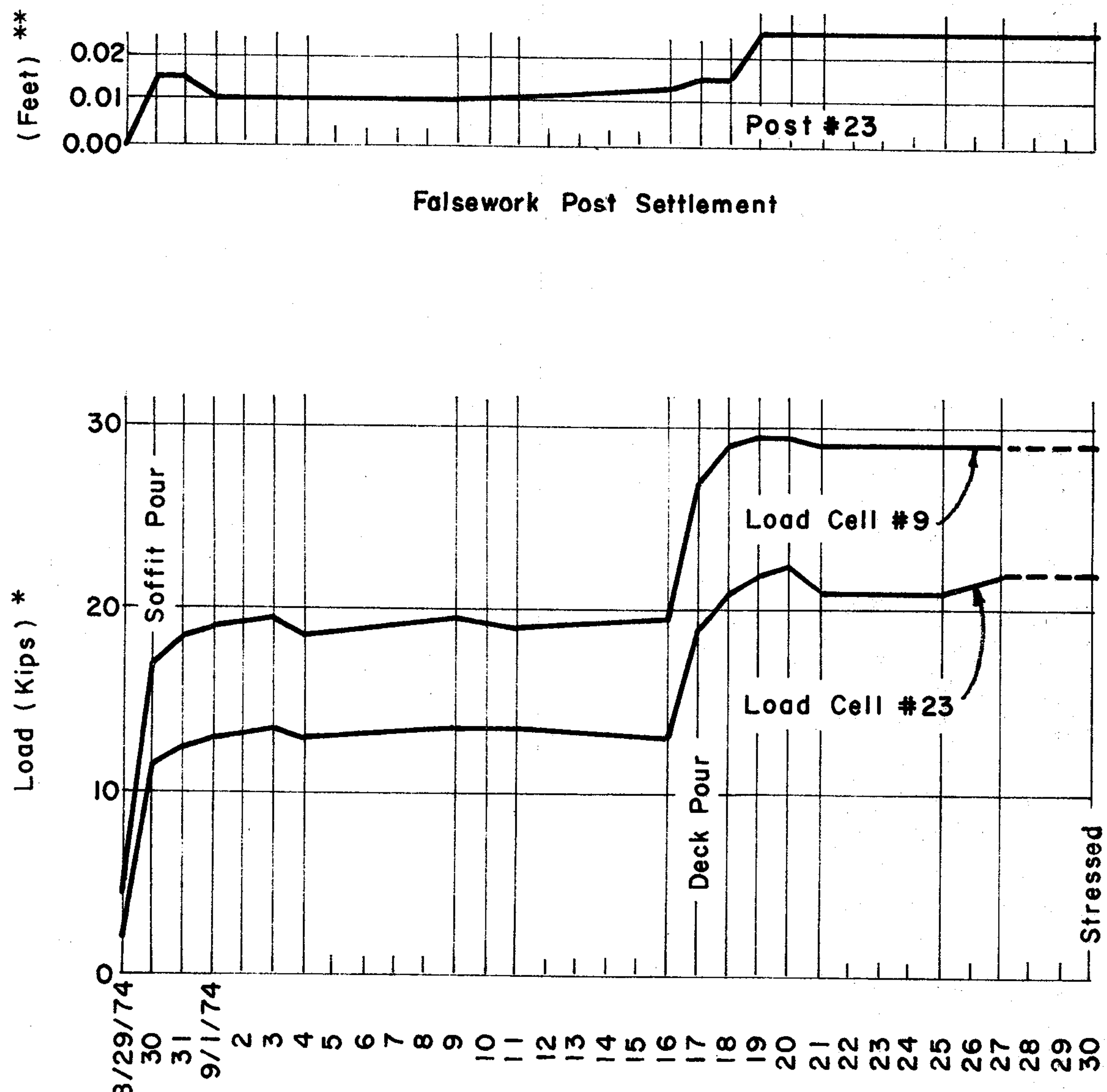
FALSEWORK LOAD CHANGES

* 1000 lb = 453 kg
** 1 ft = .303 m

Figure 38

OLD OREGON TRAIL UNDERCROSSING LEFT BRIDGE

Br. No. 6-178L

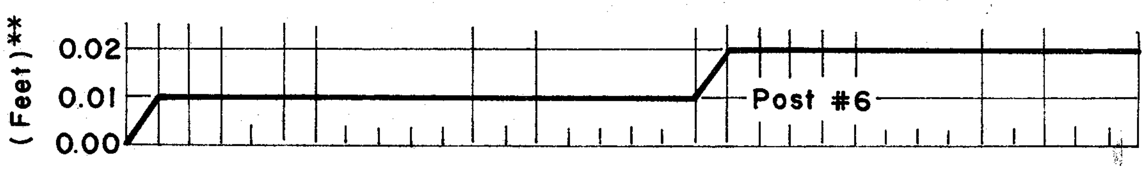


FALSEWORK LOAD CHANGES

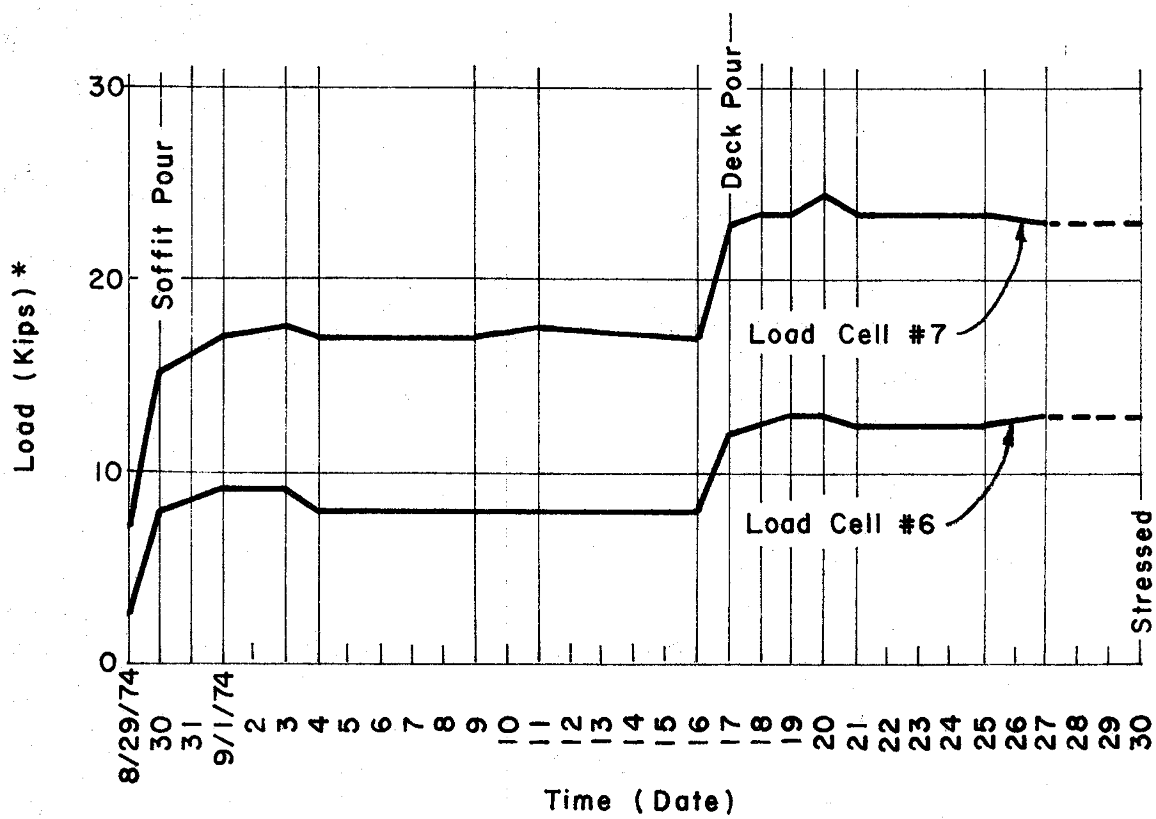
* 1000 lb = 453 kg

** 1 ft = .303 m

Figure 39



Falsework Post Settlement



FALSEWORK LOAD CHANGES

* 1000 lb = 453 kg

** 1 ft = .303 m

Figure 40

METHOD OF SAMPLING FRESH CONCRETE

Scope

This method describes the procedure for obtaining samples of fresh concrete from stationary and paving mixers, from truck mixers, agitators or dump trucks, and from forms and subgrade.

A. Size of Sample

1. When the sample will be used for strength tests, it shall be a minimum of 1 cubic foot. Smaller samples may be permitted for other routine tests.

B. Where Sample Should Be Taken

1. When sampling to determine whether the compressive strength conforms to a strength specification, take the sample as close as practicable to the mixer discharge.

2. When sampling to determine compressive strength for form stripping purposes, etc., take the sample as close as practicable to the final resting place of the concrete.

C. Procedure for Sampling

When sampling, include every precaution necessary to obtain samples that will be representative of the true nature and condition of the concrete being sampled. Sample concrete during the placing operation as follows:

1. Sampling from Stationary Mixers, Except Paving Mixers

Obtain the sample by passing a receptacle completely through the discharge stream of the mixer at about the middle of the batch, or by diverting the stream completely so that the whole stream discharges into a container. Take care not to restrict the flow from the mixer in such a manner as to cause the concrete to segregate. These requirements apply to both tilting and non-tilting mixers.

2. Sampling from Paving Mixers and from Haul Vehicles without Agitation

Discharge the concrete onto the subgrade and collect the sample from at least five different portions of the pile. Sample may be obtained after concrete has passed through a spreader box.

3. Sampling from Revolving Drum Truck Mixers or Agitators

Sample from three or more regular intervals throughout the discharge of the entire batch avoiding the very beginning and the end of the discharge. If water is added to the mixer to adjust the slump at the site of the work, sample after the water is added and the concrete is thoroughly mixed. Sample by repeatedly passing a receptacle through the entire discharge stream, or by diverting the stream completely so that the whole stream discharges into a container. Regulate the rate of discharge of the batch by the rate of revolution of the drum, and not by the size of the gate opening.

4. Sampling from Forms

Special care must be taken to obtain a representative sample. Make up the sample from several portions at different locations within the batch and at sufficient depth to include representative ingredients. Take samples prior to any finishing operations.

D. Remixing Sample

Prior to testing or molding test specimens, remix the sample with a shovel to insure uniformity. Protect the sample from sunlight and wind during the period between taking and using. Combined testing and molding time shall not exceed 15 minutes from the time of sampling.

E. Precautions

Use proper lifting methods to avoid injuries when lifting the concrete and secure assistance when needed. Be careful to keep clear of moving equipment when obtaining samples.

REFERENCES

AASHO T-141
ASTM C-172

End of Text on Calif. 539-A

METHOD OF MAKING, HANDLING, AND STORING CONCRETE COMPRESSIVE TEST SPECIMENS IN THE FIELD

Scope

The procedure for making, handling, and storage of concrete compressive strength test specimens in the field is described in this method.

Procedure

A. Apparatus

The following apparatus may be obtained from the Service and Supply Department by California Division of Highways agencies:

1. Molds—6-in. by 12-in. test cylinder molds with lids, conforming to ASTM C 470 for metal molds.
2. Tamping rod, ⅝-in. diameter by 24-in. long with a hemispherical tip at one end, or both ends, the diameter of which is ⅝-inch.
3. Suitable scoop. (A large popcorn or sugar scoop is satisfactory.)

B. Preparation of Test Specimens

1. Sampling Fresh Concrete

a. The procedure for sampling shall be as outlined in Test Method No. Calif. 539, "Method of Sampling Fresh Concrete." Write the concrete sample location (deck, footing, girder, etc.) on the sample identification card. Samples should normally contain not less than one cubic foot.

b. Transport samples in watertight containers to the place where the test specimens are to be molded. Mold specimens as near as practicable to the place where they are to be stored during the first 24 hours. If the maximum size of coarse aggregate used exceeds 2 inches, screen the concrete sample through a 2-inch sieve, discard the oversized aggregate, and note this on the sample identification card. Before molding specimens, remix the sample with a shovel to assure that there is no segregation of the coarse aggregate and mortar.

2. Molding Test Specimens

a. Place test molds on a firm, flat surface to prevent distortion of the bottom surface. When more than one specimen is to be made from the same batch, make all specimens simultaneously. Place and rod the first layer in each mold before proceeding to the second layer and so on, through the third layer. Place an approximate 4-inch layer of concrete in the mold with a circular motion of the scoop to distribute the concrete evenly in the mold. Rod the layer 25 times with the specified tamping rod, penetrating full depth into the layer, but not forcibly striking the bottom of the mold. Distribute the 25 strokes evenly over the surface of the layer. Place two additional layers in the mold, each approximately one-third of the volume of the mold, and rod each layer with 25 strokes of the tamping rod. When rodding the second layer, penetrate just into the first layer with each stroke, and also penetrate into the second when rodding the third layer. Pat sides of the mold lightly by hand, or jig by rocking can from side to side, after each layer is rodded to release any entrapped air along the sides of the mold.

b. After the top layer has been rodded and the sides of the mold patted, strike off the surface of the concrete even with the top edge of the mold. Wipe the sides of the mold free of excess concrete and press the lid on to prevent evaporation.

c. To prevent loss of moisture, seal the cover to the mold with masking tape. Do not apply water on top of the concrete before covering.

d. Clearly identify cylinders on the side of the mold with a marking pen showing the contract number, sample number, and the testing age designated.

Note:

If specimens are representative of concrete for precast products, vibration similar to that applied to the member being manufactured may be used to consolidate the specimens in lieu of the rodding procedure described in a and b.

3. Care of Test Specimens

a. Place the concrete test cylinders in their field curing location as soon as possible after they are fabricated, being careful not to disturb the concrete in its plastic state. The curing location must be a firm level surface, free from vibration and otherwise protected from disturbance. Cure all test specimens with the axis of the cylinder vertical to avoid a sloping end in the hardened concrete. Once the concrete has begun to set, do not disturb specimens for 20 ± 4 hours.

b. Handling and storage of cylinders shall conform to one of the following methods:

(1) Method 1—Cylinders for Determining the Acceptability of Concrete which has a Specified 28-day Strength.

Except for steam cured concrete, cylinders in this category shall be stored under conditions that maintain a temperature of 60° to 80°F immediately adjacent to the specimens for a period of one day. This can conveniently be achieved by the use of a water tank. At the end of 20 ± 4 hours, remove the lids from the cylinder cans and store the specimens in a water bath at a temperature of 60° to 80°F. At an age of two days and no later than five days, replace lids, resealing with masking tape, and ship directly to the laboratory.

(2) Method 2—Cylinders for Evaluating the Inplace Strength of Concrete in a Structure Prior to Applying Loads or Stresses.

For determining compressive strength under this category, store specimens at or near the structure in a semisheltered location where the temperature of the test specimens will be approximately that of the concrete in the structure. Leave the specimens at the structure for as long a period of time as possible before shipping to the laboratory. During the storage time at the structure, keep specimens in a plywood box (without insulation) or other suitable shelter, but in a shaded location. Avoid conditions of extreme exposure to wind and sun, as well as conditions of overprotection from weather variations.

(3) Method 3—Cylinders for Evaluating Steam Cured Concrete for Compliance with Strength Specifications.

Cylinders for determining time of stressing or loading shall be cured in the same manner as the member.

Cylinders for determining compliance with 28-day strength requirements shall be cured in the same manner as the member until completion of the steam curing process and then transferred to a water bath or moist room at 60° to 80°F until tested.

As an alternative to shipping to a State laboratory, testing may be done using the producer's equipment, provided that satisfactory evidence has been furnished that such equipment, together with testing procedures comply to accepted standards of testing, such as Test Method No. Calif. 521, or ASTM Designation C-39.

Note:

In lieu of molds specified under A-1, reuseable vertical molds conforming to the requirements of ASTM Designation: C-192 may be used.

REFERENCES

Test Methods Nos. Calif. 521 and 539
ASTM Designations: C-31, C-39, C-192 and C-470

End of Text on Calif. 540-B

Deflections of Simple Span
CIP Prestressed Box Girder

January 20, 1970

MEMO TO DESIGNERS:

The calculated deflection of any concrete structure involves assumptions concerning modulus of elasticity, fixity of supports, effect of skew, creep factor, section properties, etc. For prestressed structures there is added the effect of the prestress force, its initial and final value; cable path, planned and actual; freedom of structure to shorten; etc.

In order to determine if the actual and calculated deflections are in reasonable agreement, we need to have a uniform method of calculating the deflection and showing the required camber on the plans.

The Construction Department is starting a systematic method of measuring structure shortening and deflections in the field. The results of their work may modify our method of calculating deflections. In the meantime, the attached example illustrates the method to be used on simple span CIP prestressed box girders.

Until the computer solution is revised, these values will have to be calculated longhand. It can be seen from the example that in some cases the upward and downward deflections may be equal, or result in a net upward deflection. Due to the inherent uncertainties in these calculations, some downward deflection should be anticipated. It will be our practice for simple span structures to call for a minimum upward camber of .01 ft per 10 ft of span length.

For the usual diaphragm abutment, it can be shown that the deflection resulting from end moments caused by resistance of the abutment to movement is small and may be ignored.

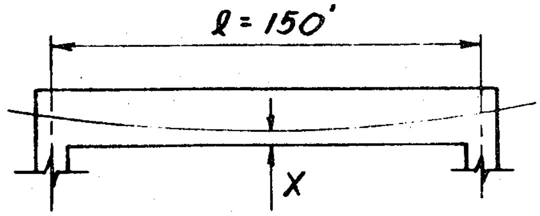
R. C. Cassano

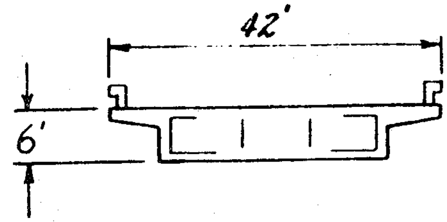
W. J. Jurkovich

Attach
RCB:bt

EXAMPLE CALCULATION
SIMPLE SPAN CIP PRESTRESSED BOX GIRDER

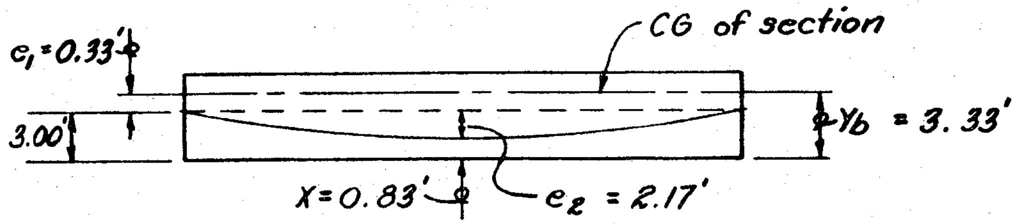
GIVEN:





$X = 8''$ $P_f = 8{,}800{,}000$ lbs.
$= 10''$ $= 9{,}200{,}000$
$= 12''$ $= 9{,}600{,}000$

Area = 65.5 sf
$I = 348\ ft^4$
$Y_b = 3.33\ ft$
$W = 10{,}300$ lbs./ft. incl. rail



ASSUME: $E = 3.5 \times 10^6$ psi $= 504 \times 10^6$ psf
"X" = 10" $P_f = 9{,}200{,}000$ lbs., use middle value
Coef of creep, etc. = 3.0 for ultimate deflection

CALCULATE:

1. Deflection caused by the <u>final</u> prestressed force

$$\Delta_{P_f} = \frac{P_f\,\ell^2}{48\,EI}\,(5e_2 + 6e_1)$$ This sign is minus if the CG of anchorages is above Y_b

$$\Delta_{P_f} = \frac{9{,}200{,}000 \times \overline{150}^2}{48 \times 504 \times 10^6 \times 348}\,(5 \times 2.17 + 6 \times .33) = 0.32\ ft. \uparrow$$

2. Deflection due to dead loads

$$\Delta_{DL} = \frac{5w\ell^4}{384\,EI} = \frac{5 \times 10{,}300 \times \overline{150}^4}{384 \times 504 \times 10^6 \times 348} = 0.39\ ft \downarrow$$

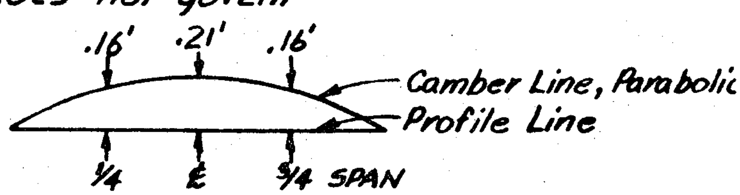
3. Ultimate theoretical deflection

$$\Delta = 3.0\,(0.39 - 0.32) = 0.21\ ft \downarrow$$

4. Check for required minimum camber

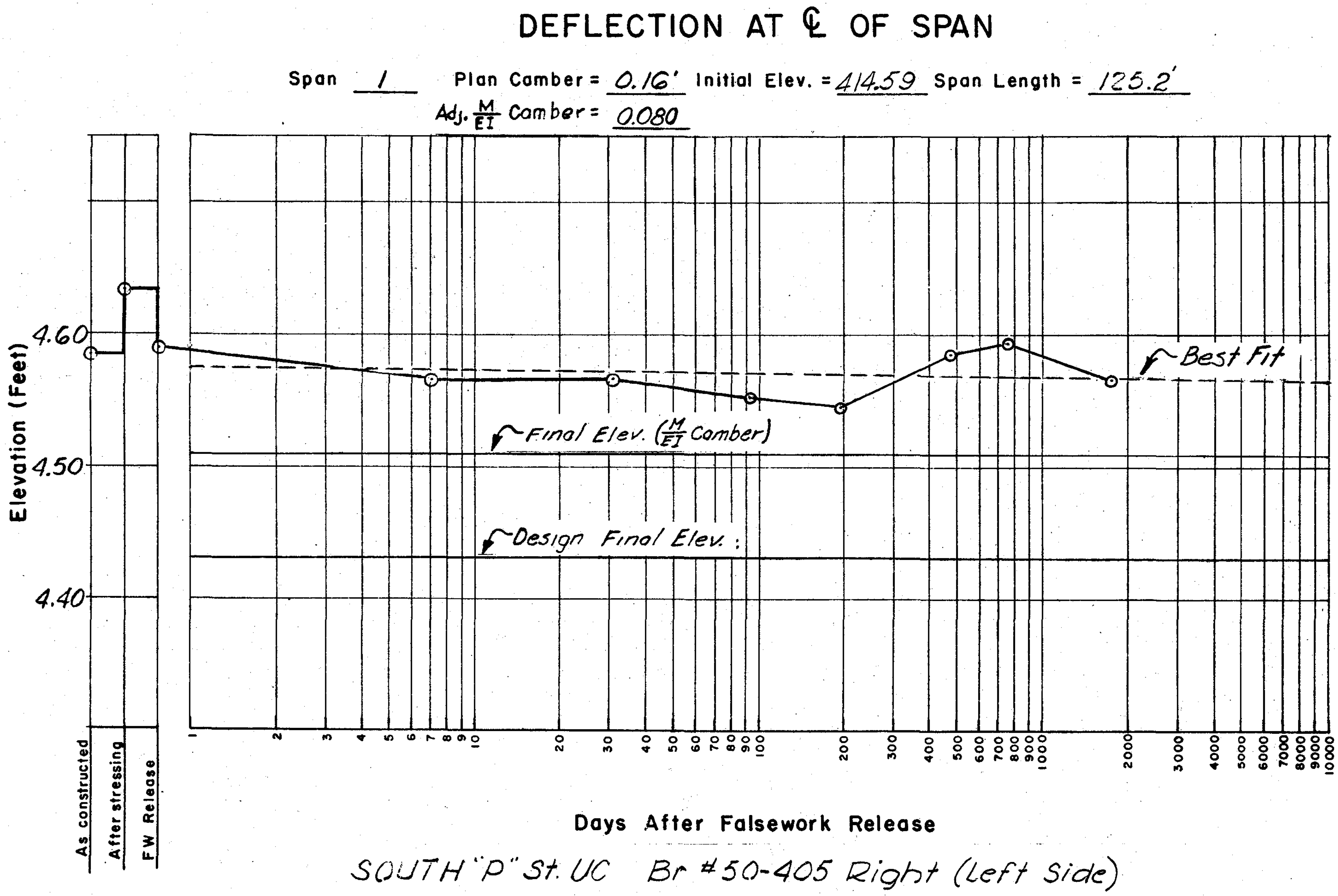
$$\frac{150}{10} \times 0.01' = 0.15 \uparrow$$ does not govern

SHOW ON PLANS:



CAMBER DIAGRAM

Does not include allowance for falsework settlement



DEFLECTION AT ℄ OF SPAN
Span 1 Plan Camber = 0.16' Initial Elev. = 414.59 Span Length = 125.2'
Adj. M/EI Camber = 0.080
Elevation (Feet)
4.60
4.50
4.40
Best Fit
Final Elev. (M/EI Camber)
Design Final Elev.
As constructed
After stressing
FW Release
1 2 3 4 5 6 7 8 9 10 20 30 40 50 60 70 80 90 100 200 300 400 500 600 700 800 900 1000 2000 3000 4000 5000 6000 7000 8000 9000 10000
Days After Falsework Release
SOUTH "P" St. UC Br #50-405 Right (Left Side)

Figure 41

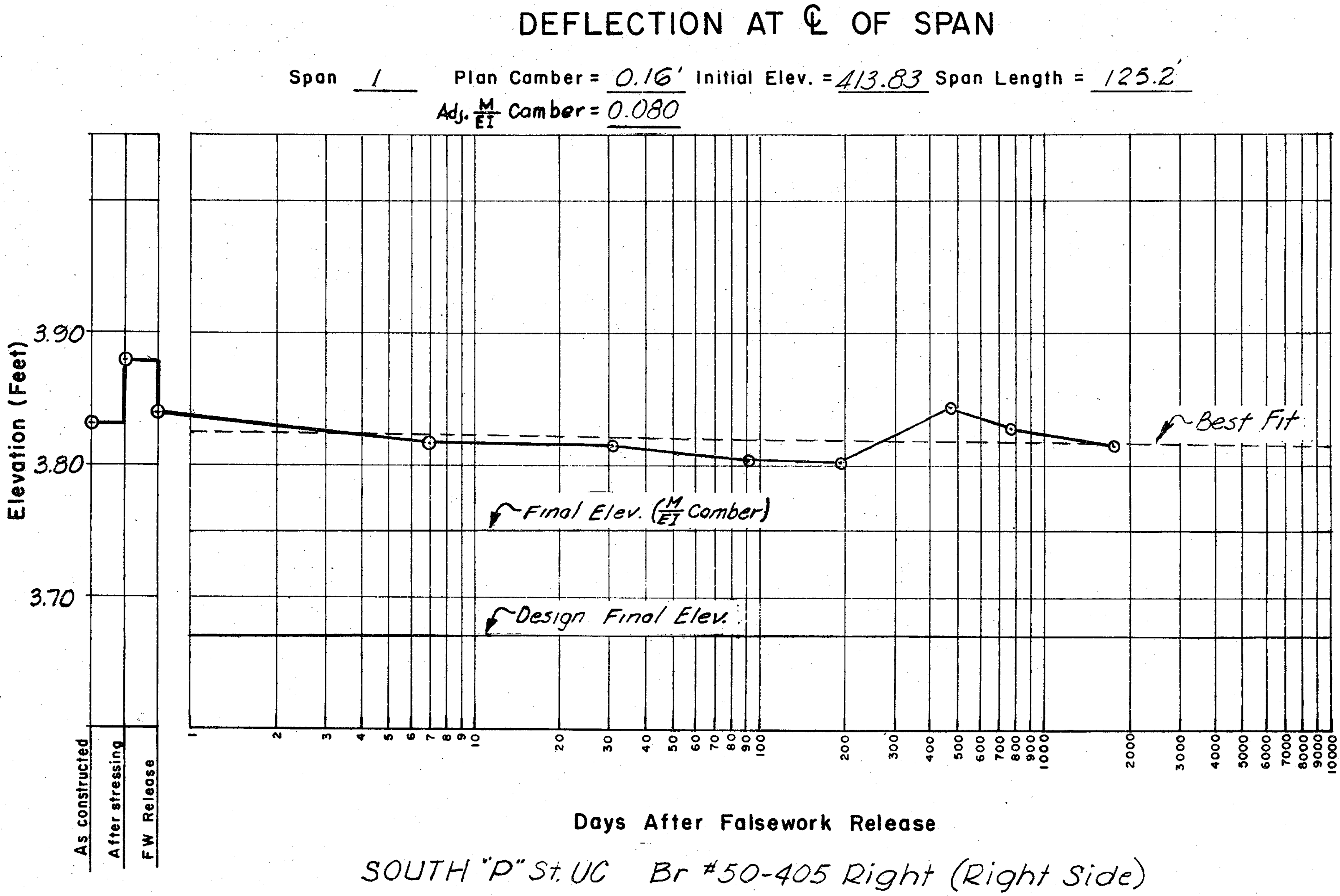
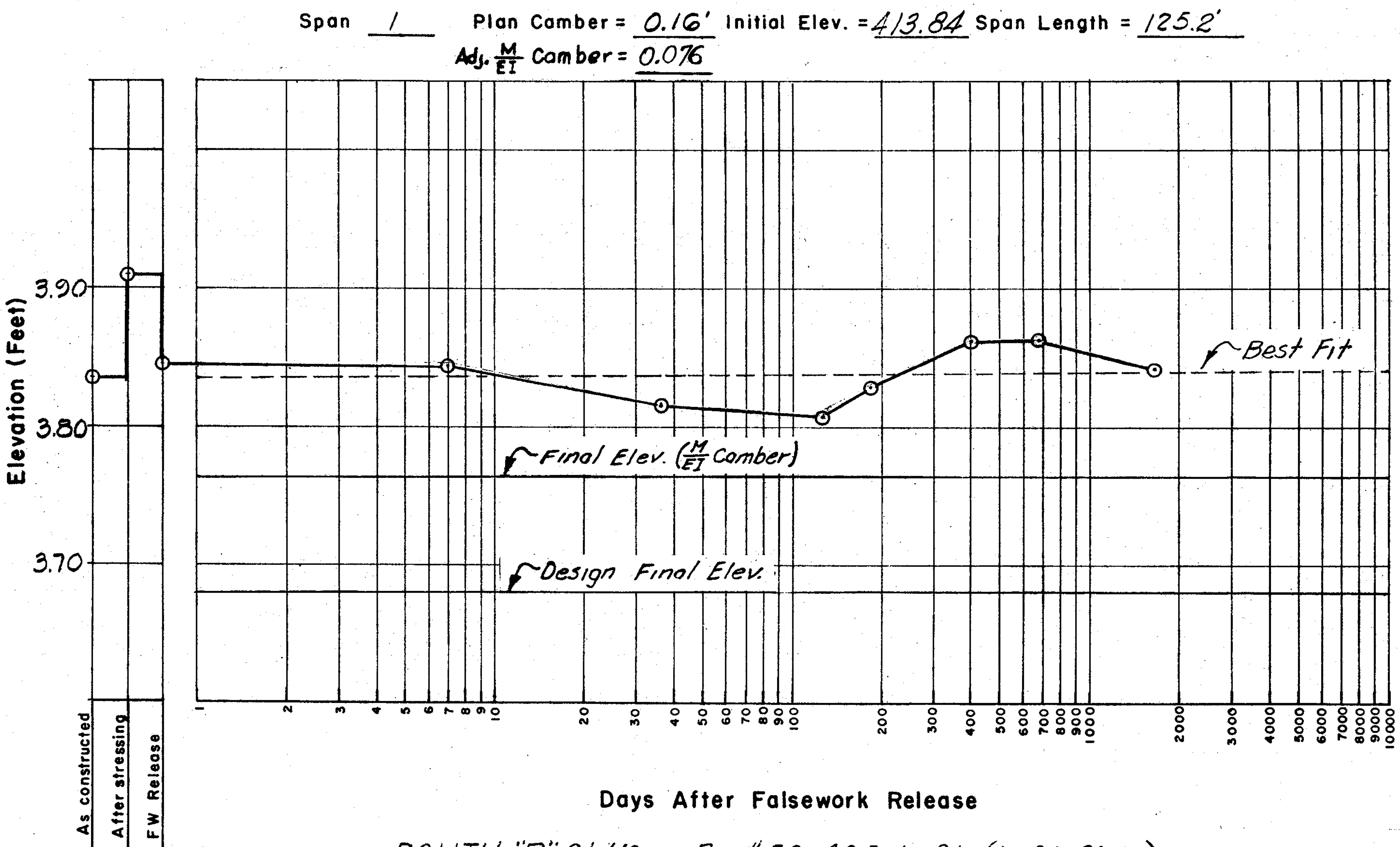

DEFLECTION AT ℄ OF SPAN
Span 1 Plan Camber = 0.16' Initial Elev. = 413.83 Span Length = 125.2'
Adj. M/EI Camber = 0.080
Elevation (Feet)
3.90
3.80
3.70
Best Fit
Final Elev. (M/EI Camber)
Design Final Elev.
As constructed
After stressing
FW Release
1 2 3 4 5 6 7 8 9 10 20 30 40 50 60 70 80 90 100 200 300 400 500 600 700 800 900 1000 2000 3000 4000 5000 6000 7000 8000 9000 10000
Days After Falsework Release
SOUTH "P" St. UC Br #50-405 Right (Right Side)

Figure 42

DEFLECTION AT ℄ OF SPAN



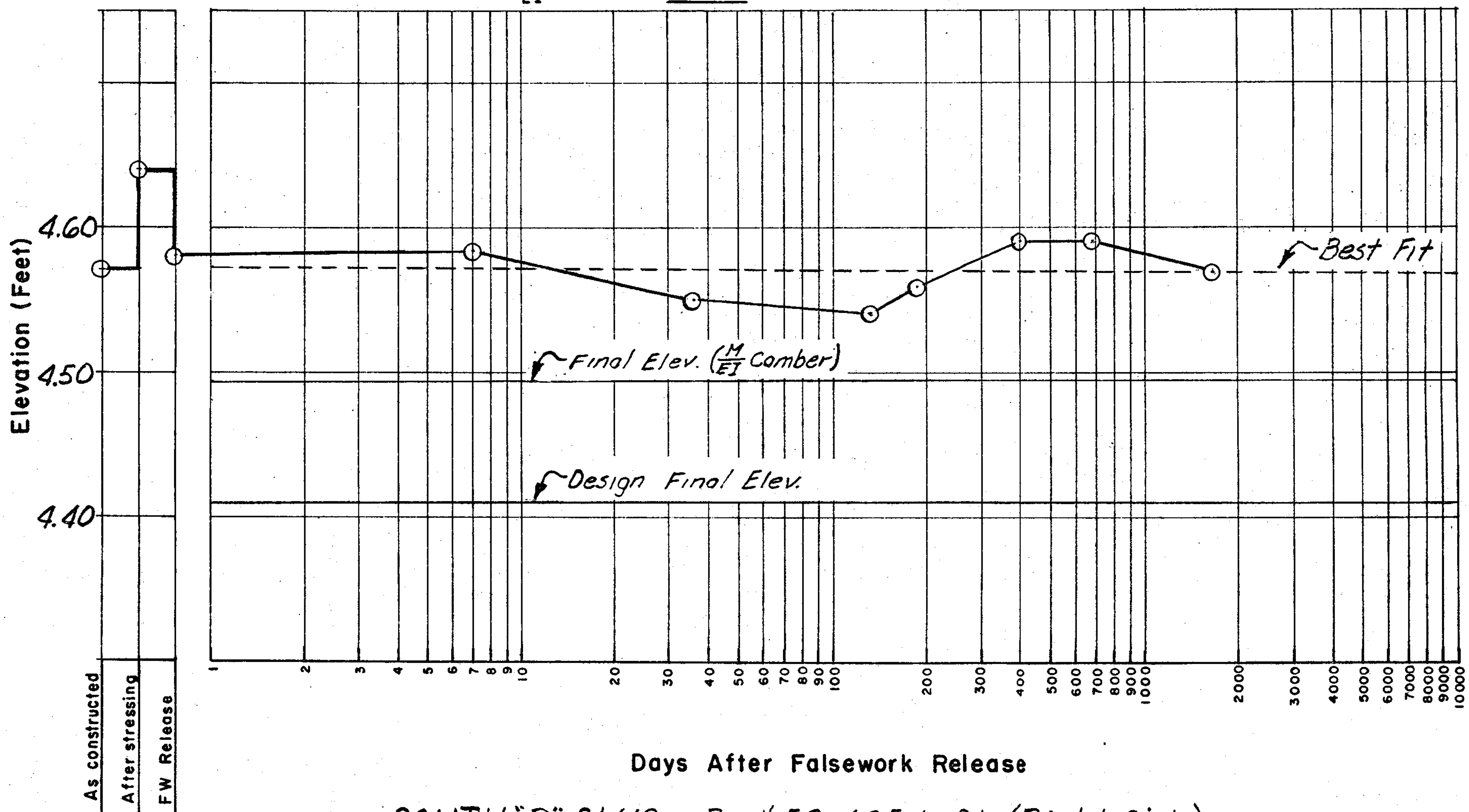
SOUTH "P" St. UC Br #50-405 Left (Left Side)

Figure 43

DEFLECTION AT ℄ OF SPAN

Span 1 Plan Camber = 0.16' Initial Elev. = 414.57 Span Length = 125.2'

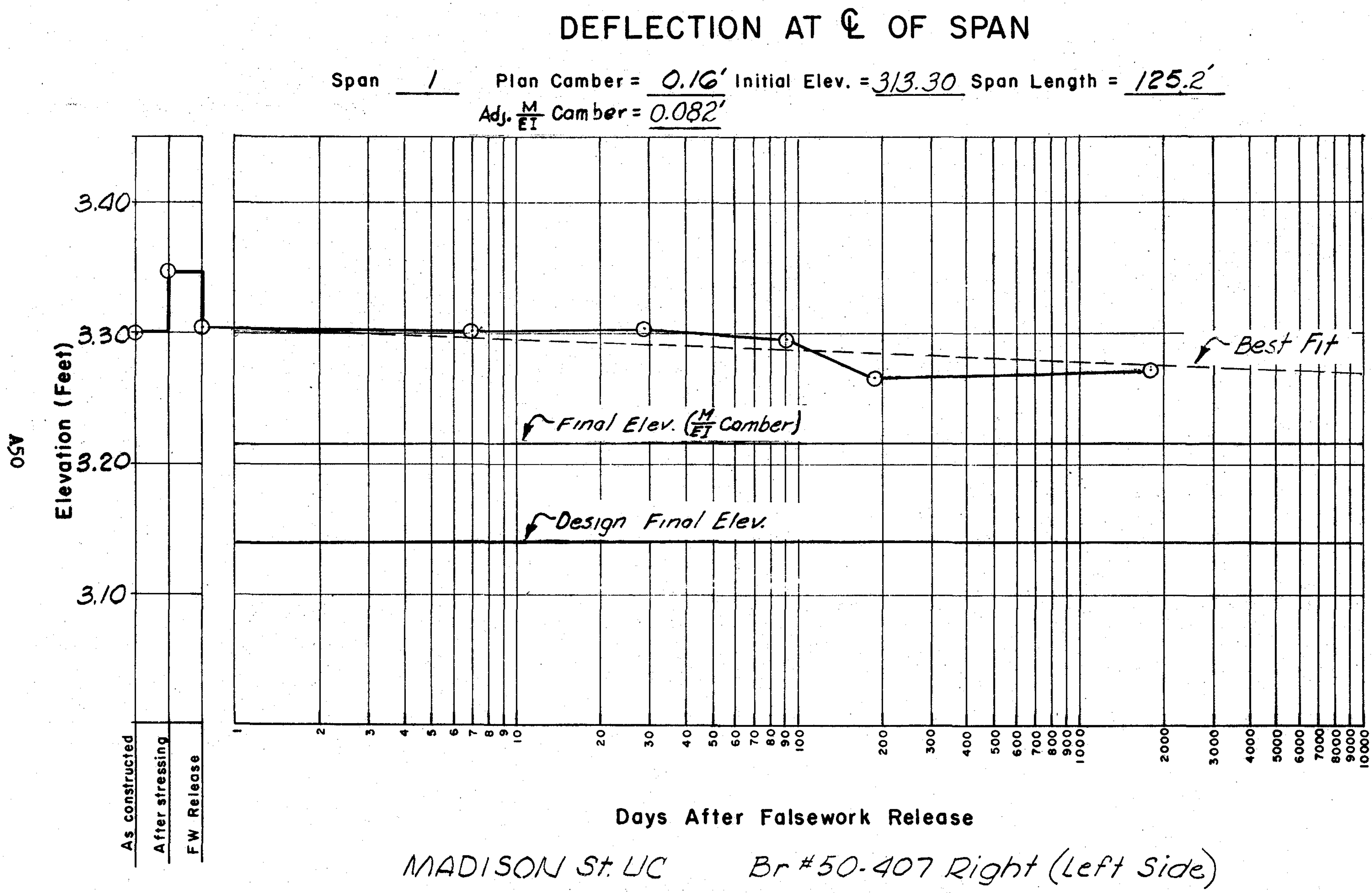
Adj. $\frac{M}{EI}$ Camber = 0.076



SOUTH "P" St. UC Br #50-405 Left (Right Side)

A49

Figure 44



DEFLECTION AT ℄ OF SPAN
Span 1 Plan Camber = 0.16′ Initial Elev. = 313.30 Span Length = 125.2′
Adj. M/EI Camber = 0.082′
A50
Elevation (Feet)
3.40
3.30
3.20
3.10
Best Fit
Final Elev. (M/EI Camber)
Design Final Elev.
As constructed
After stressing
FW Release
1
2
3
4
5
6
7
8
9
10
20
30
40
50
60
70
80
90
100
200
300
400
500
600
700
800
900
1000
2000
3000
4000
5000
6000
7000
8000
9000
10000
Days After Falsework Release
MADISON St. UC Br #50-407 Right (Left Side)

Figure 45

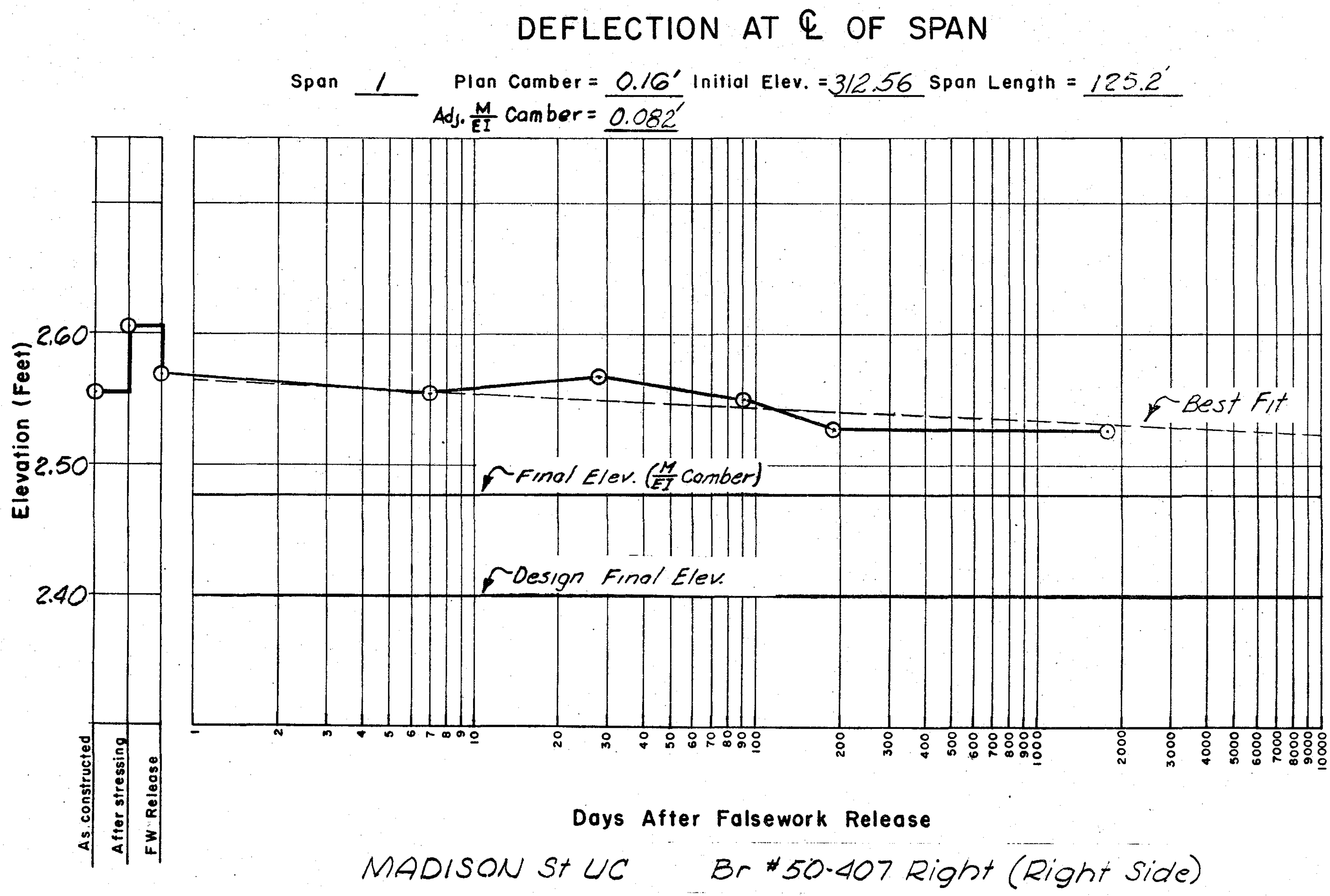
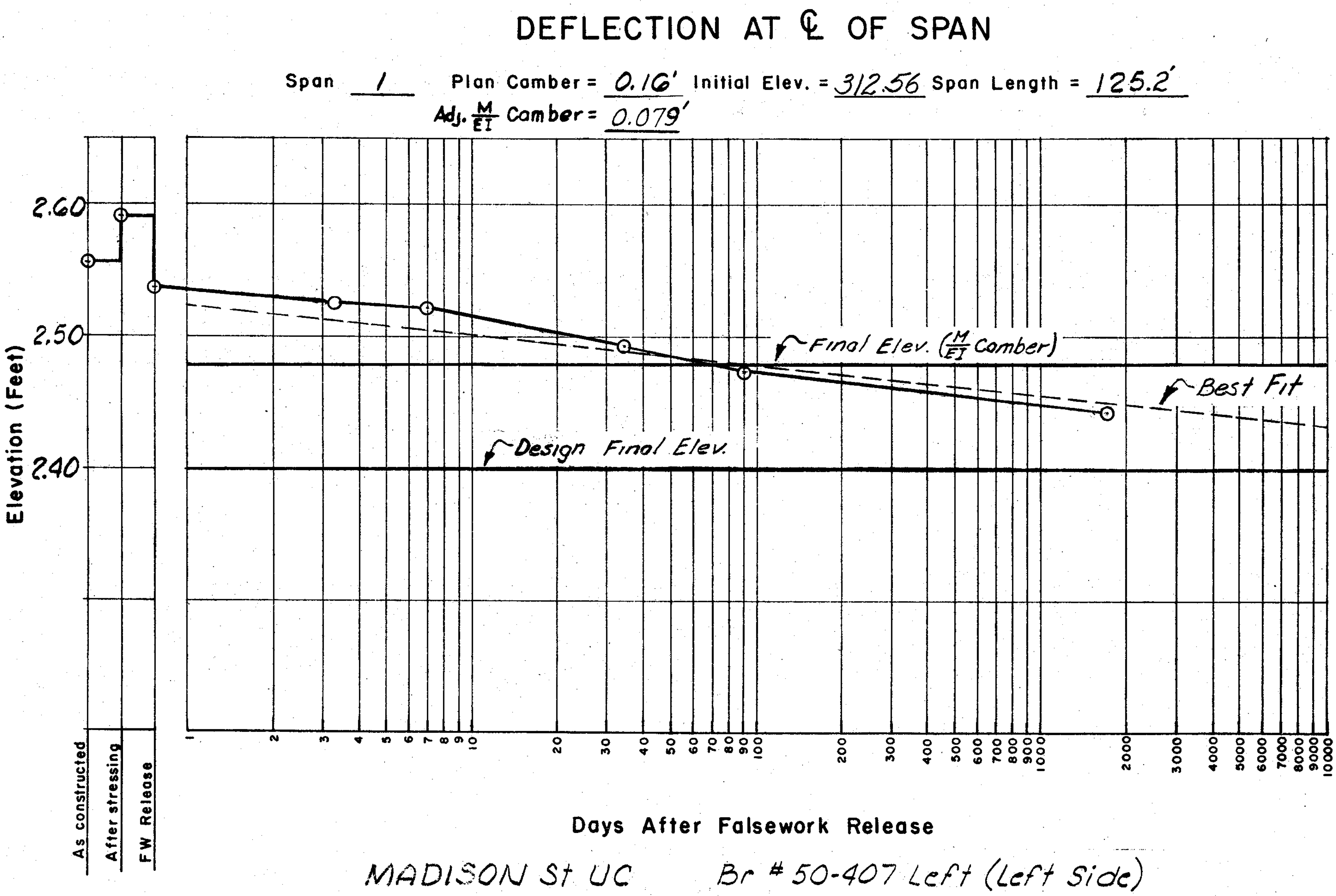

DEFLECTION AT ℄ OF SPAN
Span 1 Plan Camber = 0.16' Initial Elev. = 312.56 Span Length = 125.2'
Adj. M/EI Camber = 0.082'
Elevation (Feet)
2.60
2.50
2.40
Best Fit
Final Elev. (M/EI Camber)
Design Final Elev.
As constructed
After stressing
FW Release
1
2
3
4
5
6
7
8
9
10
20
30
40
50
60
70
80
90
100
200
300
400
500
600
700
800
900
1000
2000
3000
4000
5000
6000
7000
8000
9000
10000
Days After Falsework Release
MADISON St UC Br #50-407 Right (Right Side)

Figure 46



DEFLECTION AT ℄ OF SPAN
Span 1 Plan Camber = 0.16' Initial Elev. = 312.56 Span Length = 125.2'
Adj. M/EI Camber = 0.079'
2.60
2.50
2.40
Elevation (Feet)
Final Elev. (M/EI Camber)
Best Fit
Design Final Elev.
As constructed
After stressing
FW Release
1
2
3
4
5
6
7
8
9
10
20
30
40
50
60
70
80
90
100
200
300
400
500
600
700
800
900
1000
2000
3000
4000
5000
6000
7000
8000
9000
10000
Days After Falsework Release
MADISON St UC Br # 50-407 Left (Left Side)

Figure 47

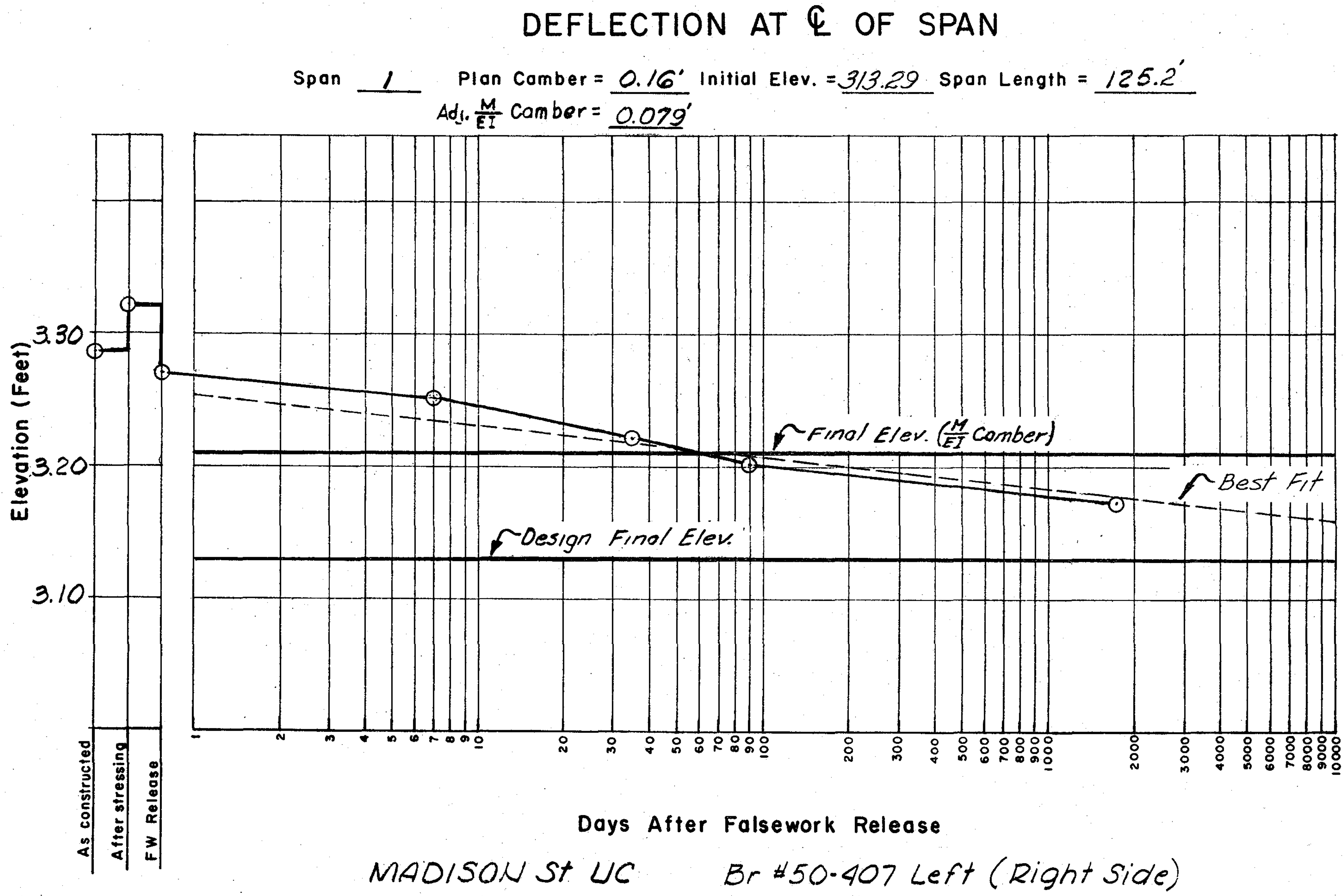

DEFLECTION AT ℄ OF SPAN
Span 1 Plan Camber = 0.16' Initial Elev. = 313.29 Span Length = 125.2'
Adj. M/EI Camber = 0.079'
Elevation (Feet)
3.30
3.20
3.10
Final Elev. (M/EI Camber)
Best Fit
Design Final Elev.
As constructed
After stressing
FW Release
1 2 3 4 5 6 7 8 9 10 20 30 40 50 60 70 80 90 100 200 300 400 500 600 700 800 900 1000 2000 3000 4000 5000 6000 7000 8000 9000 10000
Days After Falsework Release
MADISON St UC Br #50-407 Left (Right Side)

Figure 48

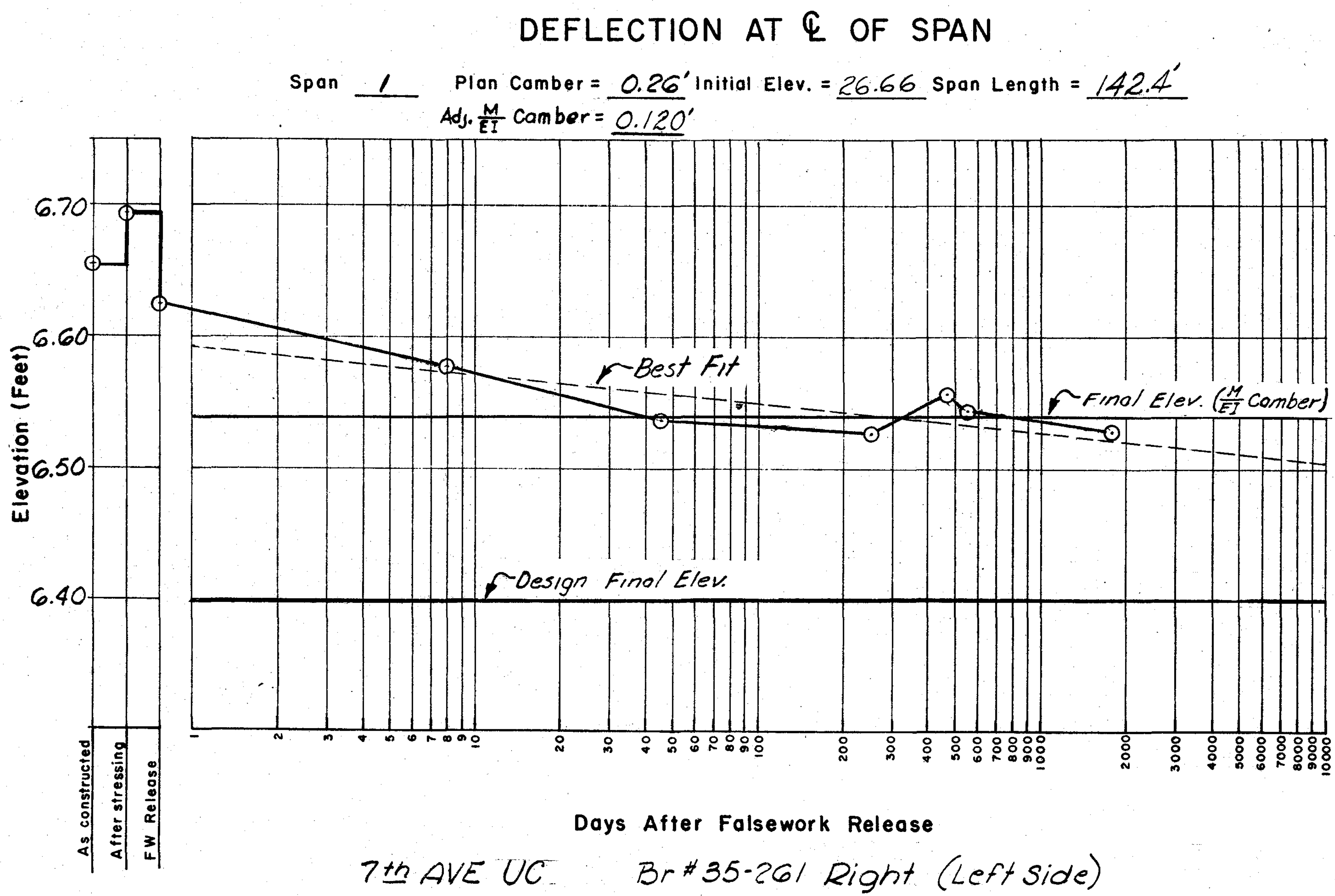
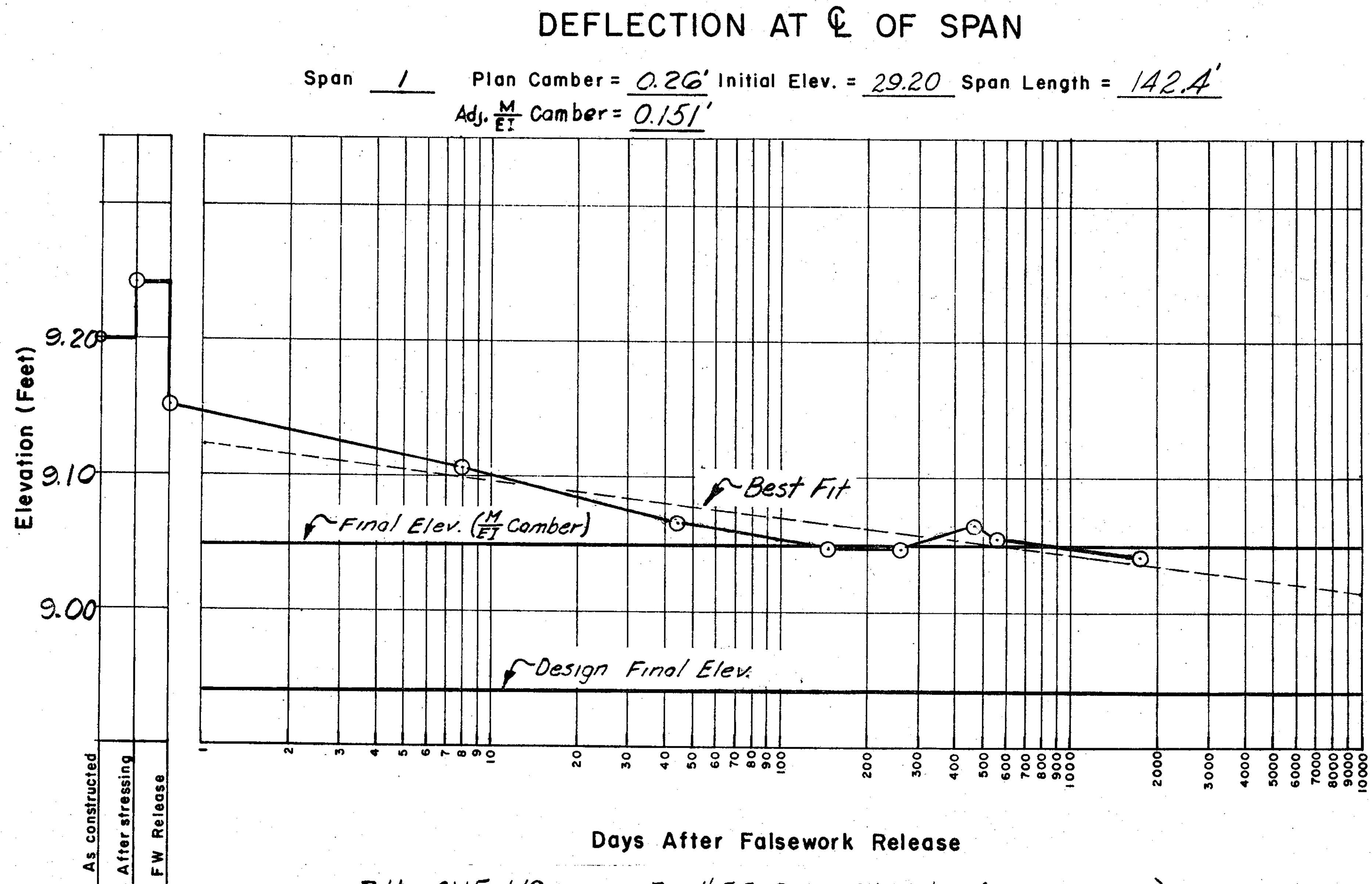

DEFLECTION AT ℄ OF SPAN
Span 1 Plan Camber = 0.26' Initial Elev. = 26.66 Span Length = 142.4'
Adj. M/EI Camber = 0.120'
Elevation (Feet)
6.70
6.60
6.50
6.40
Best Fit
Final Elev. (M/EI Camber)
Design Final Elev.
As constructed
After stressing
FW Release
1
2
3
4
5
6
7
8
9
10
20
30
40
50
60
70
80
90
100
200
300
400
500
600
700
800
900
1000
2000
3000
4000
5000
6000
7000
8000
9000
10000
Days After Falsework Release
7th AVE UC Br # 35-261 Right (Left Side)

Figure 49



A55

Figure 50

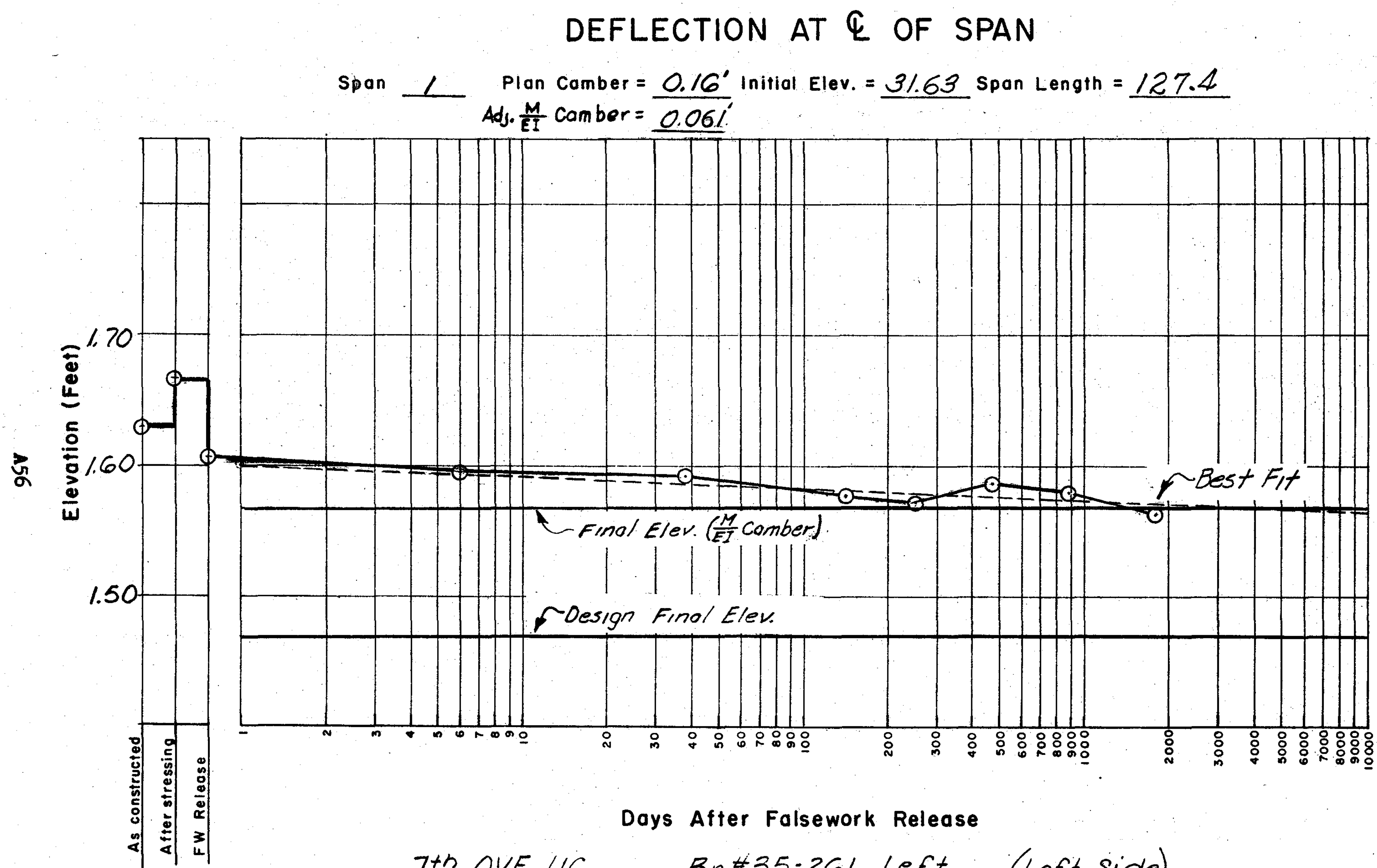
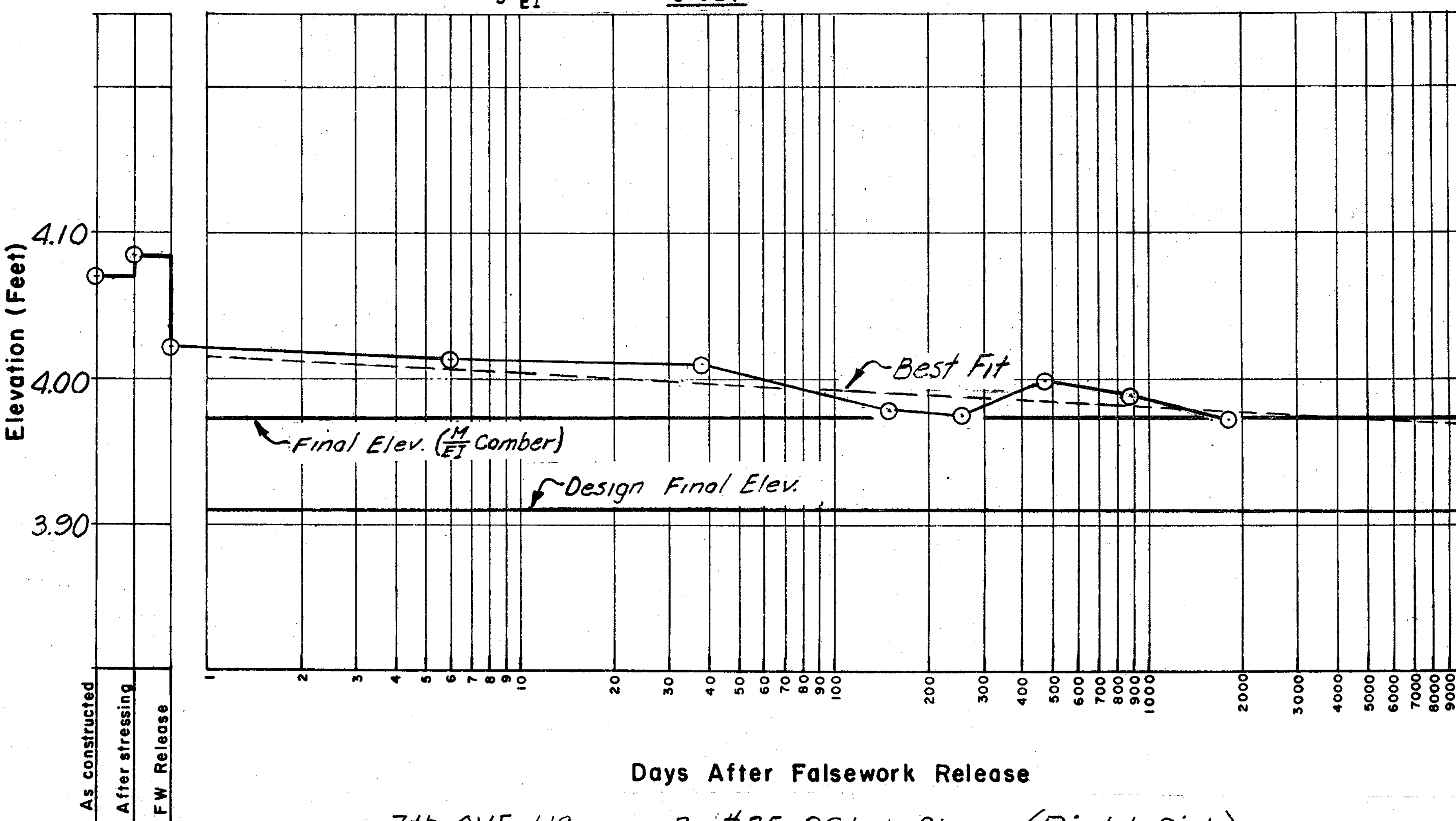

DEFLECTION AT ℄ OF SPAN
Span 1 Plan Camber = 0.16' Initial Elev. = 31.63 Span Length = 127.4
Adj. M/EI Camber = 0.061'
Elevation (Feet)
1.70
1.60
1.50
As constructed
After stressing
FW Release
Best Fit
Final Elev. (M/EI Camber)
Design Final Elev.
1 2 3 4 5 6 7 8 9 10 20 30 40 50 60 70 80 90 100 200 300 400 500 600 700 800 900 1000 2000 3000 4000 5000 6000 7000 8000 9000 10000
Days After Falsework Release
7th AVE UC Br #35-261 Left (Left Side)
A56

Figure 51

DEFLECTION AT ℄ OF SPAN

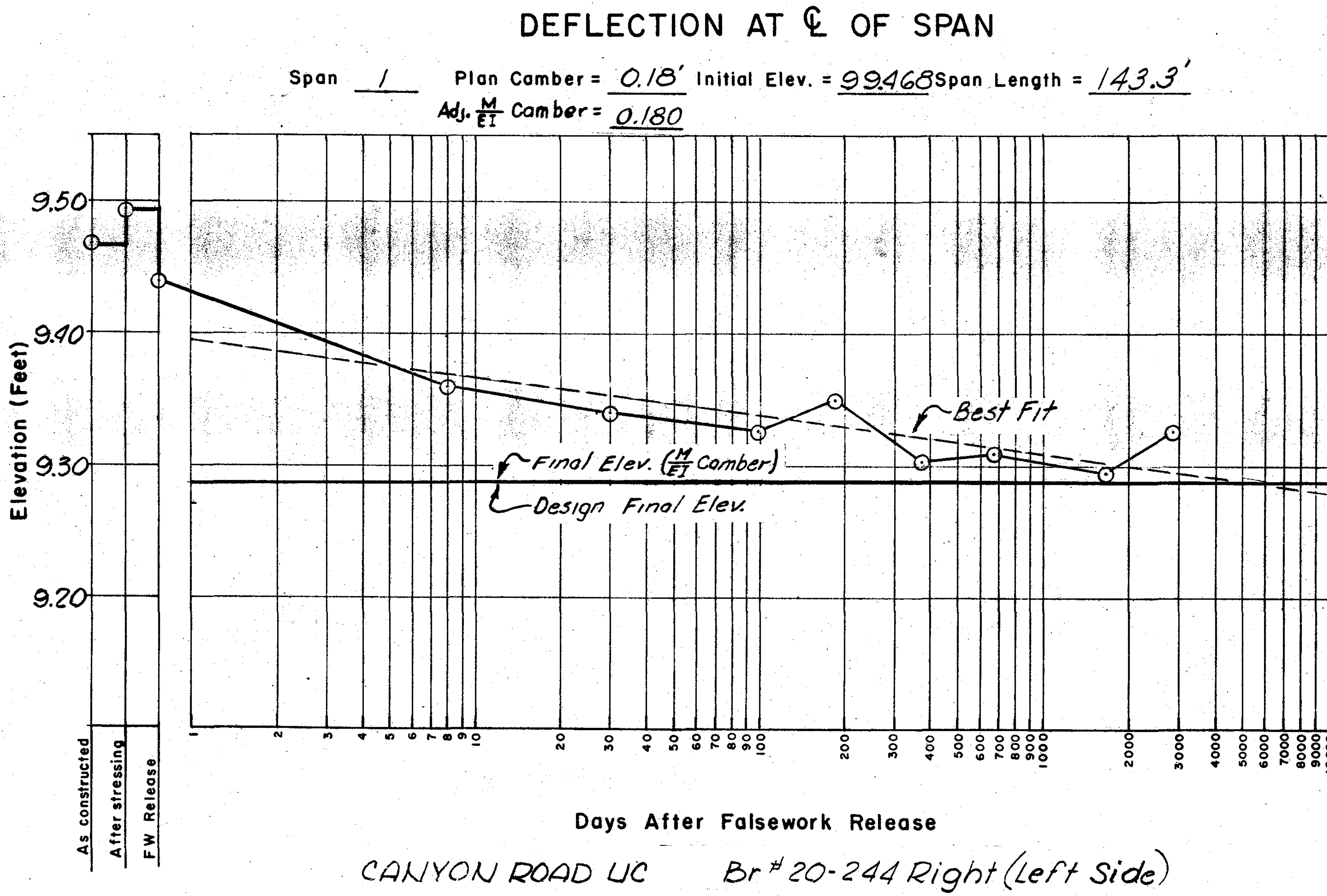
Span 1 Plan Camber = 0.16′ Initial Elev. = 34.07 Span Length = 127.4′

Adj. $\frac{M}{EI}$ Camber = 0.097



7th AVE UC Br #35-261 Left (Right Side)

Figure 52



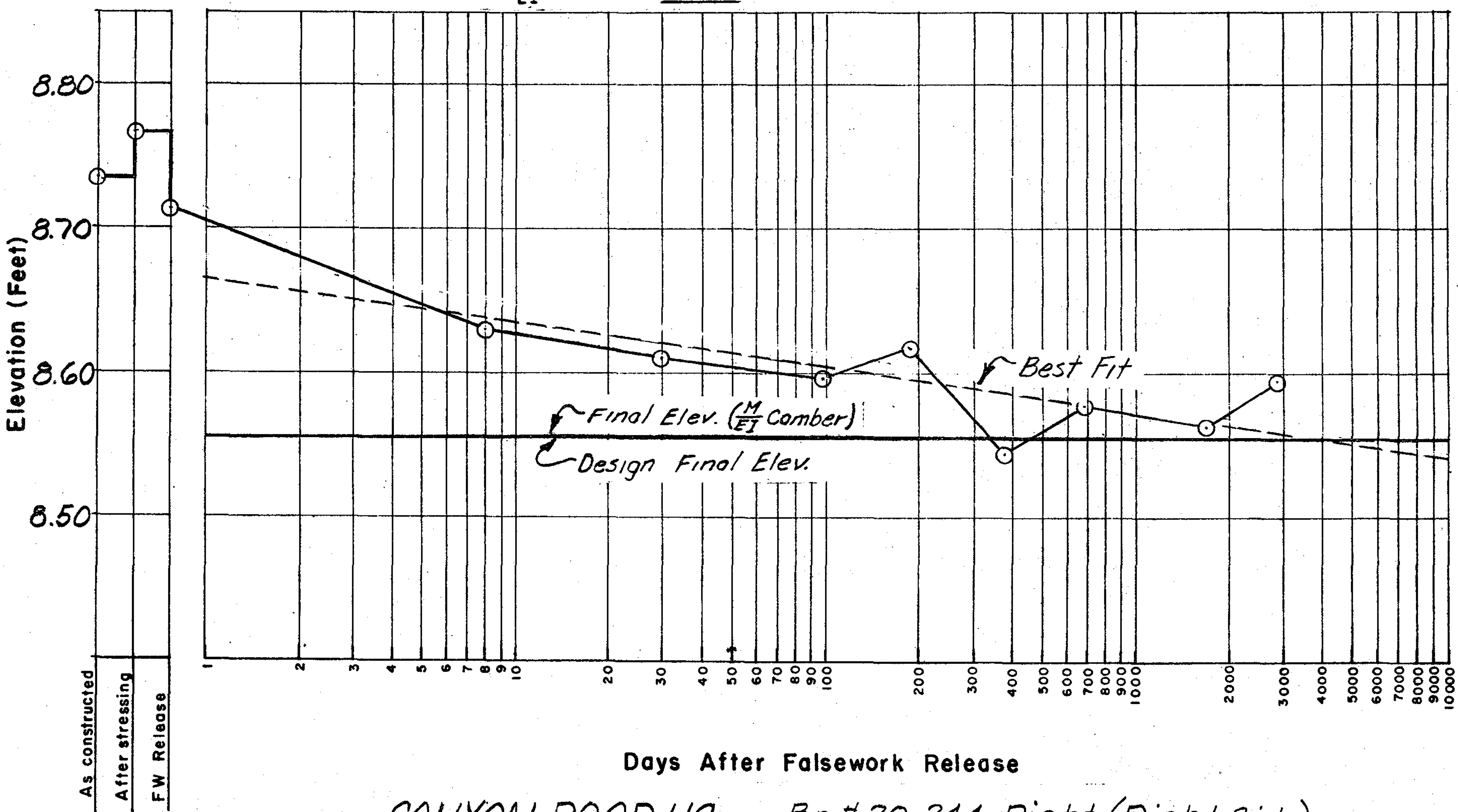
DEFLECTION AT ℄ OF SPAN
Span 1 Plan Camber = 0.18' Initial Elev. = 99.468 Span Length = 143.3'
Adj. M/EI Camber = 0.180
9.50
9.40
9.30
9.20
Elevation (Feet)
Best Fit
Final Elev. (M/EI Camber)
Design Final Elev.
As constructed
After stressing
FW Release
1
2
3
4
5
6
7
8
9
10
20
30
40
50
60
70
80
90
100
200
300
400
500
600
700
800
900
1000
2000
3000
4000
5000
6000
7000
8000
9000
10000
Days After Falsework Release
CANYON ROAD UC Br # 20-244 Right (Left Side)

Figure 53

DEFLECTION AT ℄ OF SPAN

Span 1 Plan Camber = 0.18' Initial Elev. = 98.736 Span Length = 143.3'

Adj. $\frac{M}{EI}$ Camber = 0.180



CANYON ROAD UC Br # 20-244 Right (Right Side)

Figure 54

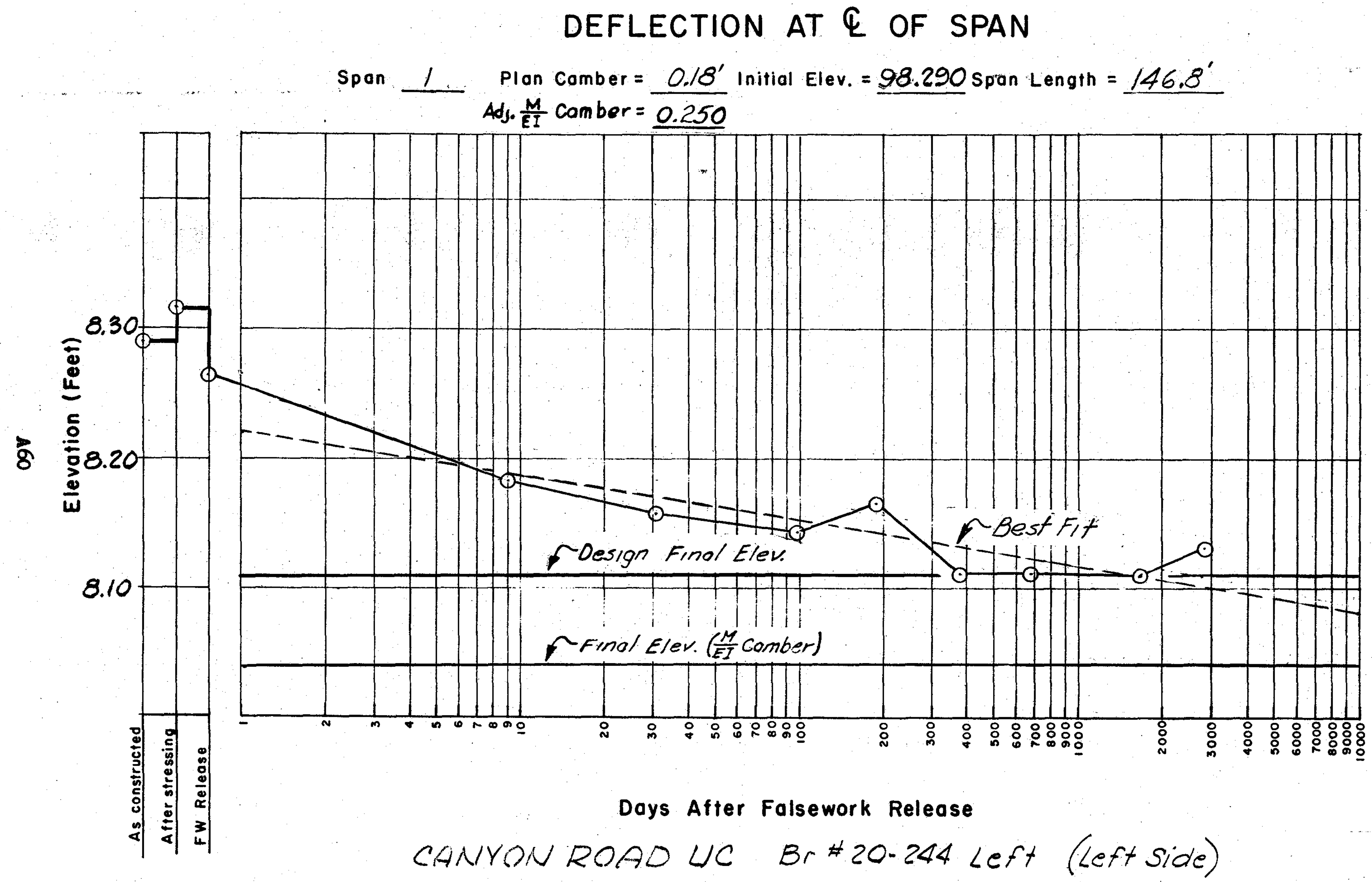

DEFLECTION AT ℄ OF SPAN
Span 1 Plan Camber = 0.18' Initial Elev. = 98.290 Span Length = 146.8'
Adj. M/EI Camber = 0.250
AV60
Elevation (Feet)
8.30
8.20
8.10
Best Fit
Design Final Elev.
Final Elev. (M/EI Camber)
As constructed
After stressing
FW Release
1
2
3
4
5
6
7
8
9
10
20
30
40
50
60
70
80
90
100
200
300
400
500
600
700
800
900
1000
2000
3000
4000
5000
6000
7000
8000
9000
10000
Days After Falsework Release
CANYON ROAD UC Br #20-244 Left (Left Side)

Figure 55

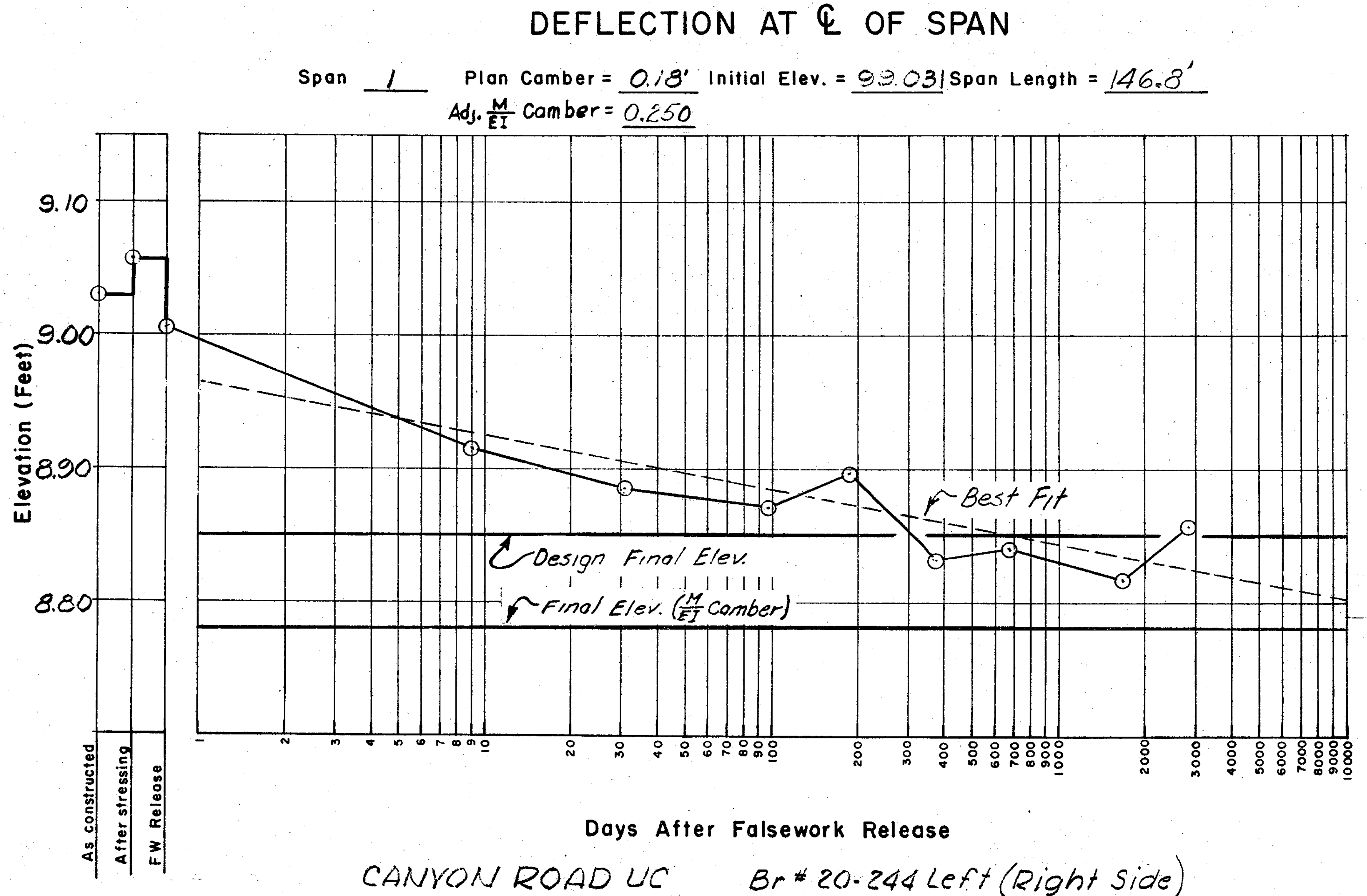

DEFLECTION AT ℄ OF SPAN
Span 1 Plan Camber = 0.18' Initial Elev. = 99.031 Span Length = 146.8'
Adj. M/EI Camber = 0.250
Elevation (Feet)
9.10
9.00
8.90
8.80
Best Fit
Design Final Elev.
Final Elev. (M/EI Camber)
As constructed
After stressing
FW Release
1
2
3
4
5
6
7
8
9
10
20
30
40
50
60
70
80
90
100
200
300
400
500
600
700
800
900
1000
2000
3000
4000
5000
6000
7000
8000
9000
10000
Days After Falsework Release
CANYON ROAD UC Br # 20-244 Left (Right Side)

Figure 56

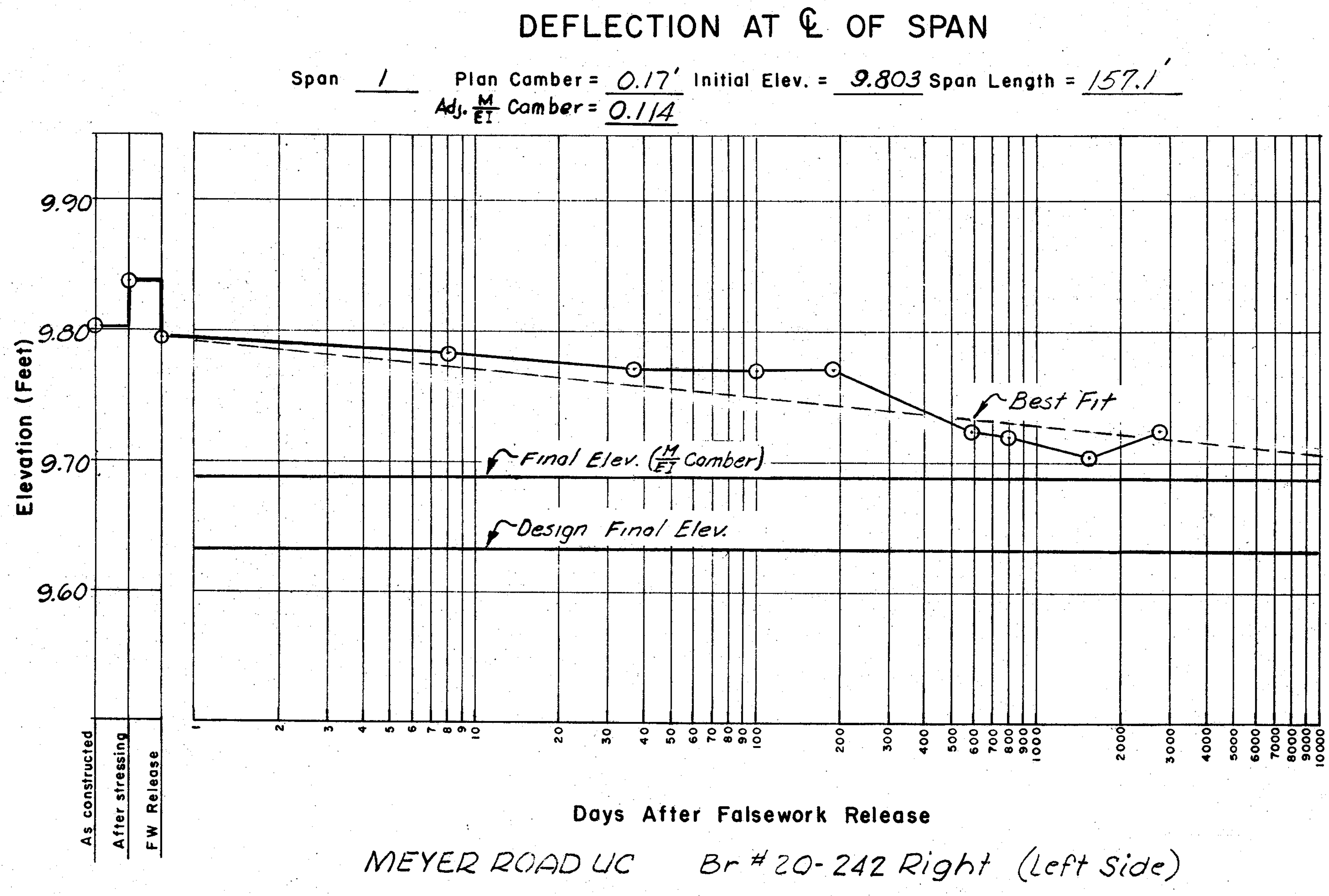

DEFLECTION AT ℄ OF SPAN
Span 1 Plan Camber = 0.17' Initial Elev. = 9.803 Span Length = 157.1'
Adj. M/EI Camber = 0.114
Elevation (Feet)
9.90
9.80
9.70
9.60
Best Fit
Final Elev. (M/EI Camber)
Design Final Elev.
As constructed
After stressing
FW Release
1
2
3
4
5
6
7
8
9
10
20
30
40
50
60
70
80
90
100
200
300
400
500
600
700
800
900
1000
2000
3000
4000
5000
6000
7000
8000
9000
10000
Days After Falsework Release
MEYER ROAD UC Br # 20-242 Right (Left Side)

Figure 57

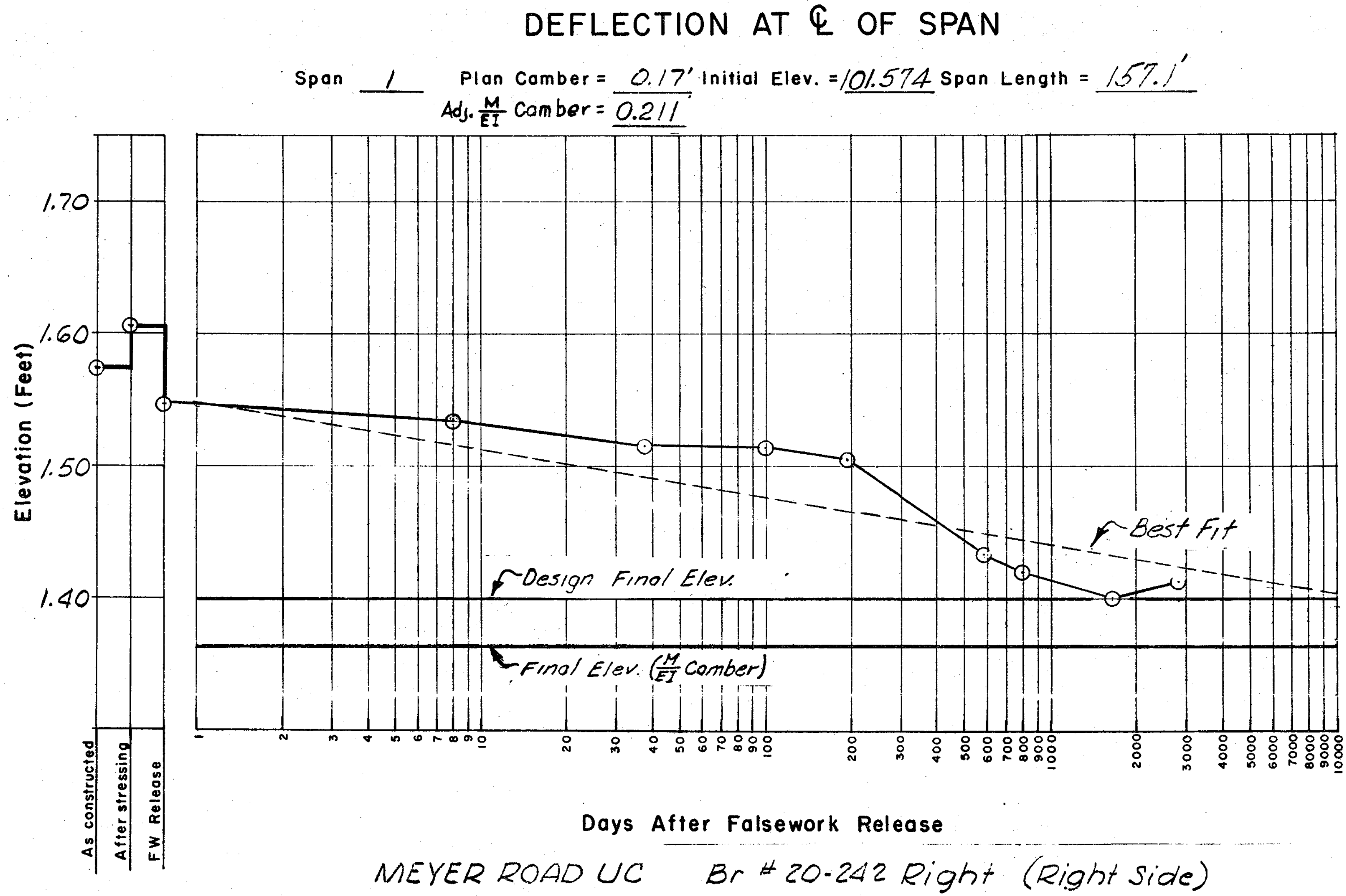
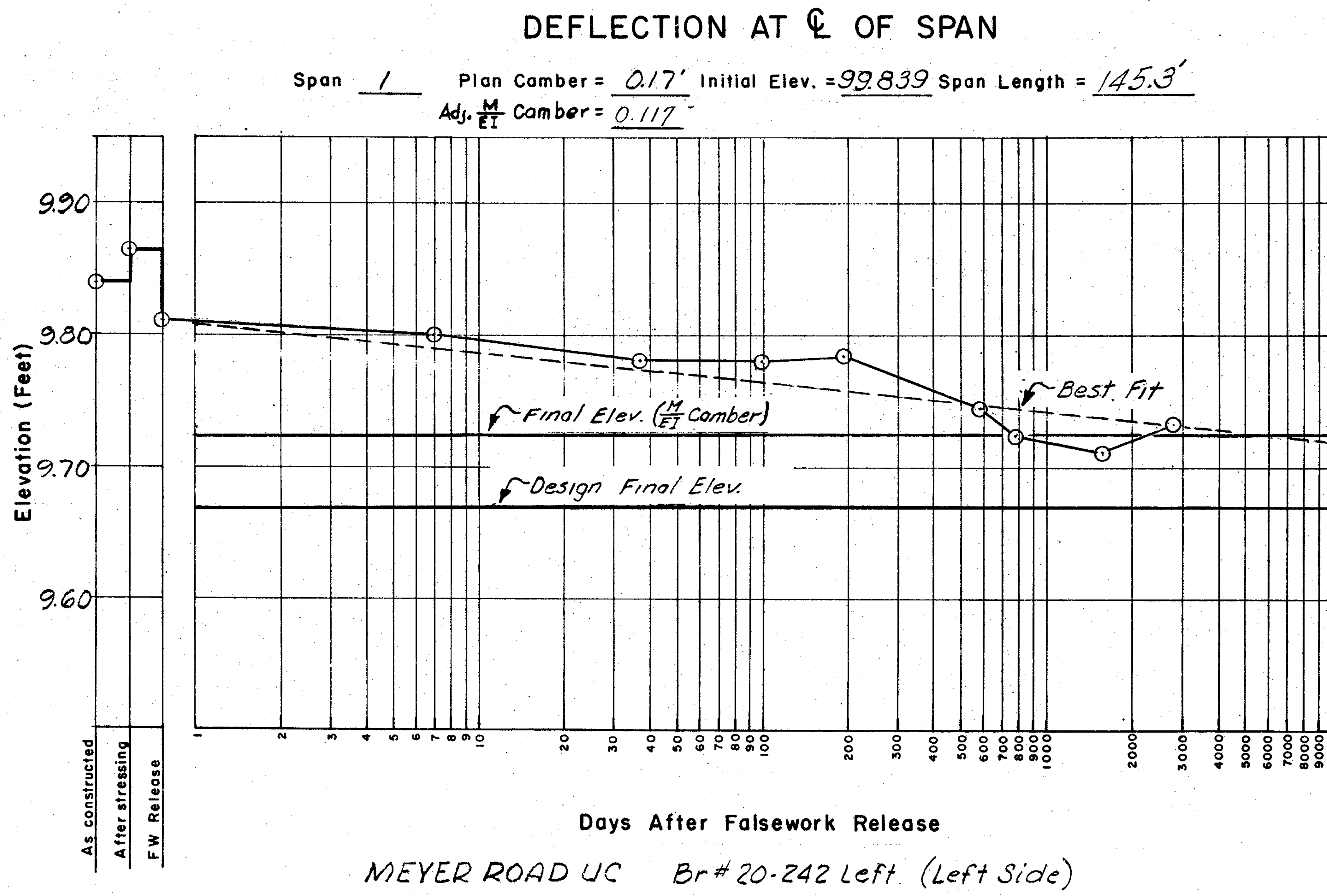

DEFLECTION AT ℄ OF SPAN
Span 1 Plan Camber = 0.17' Initial Elev. = 101.574 Span Length = 157.1'
Adj. M/EI Camber = 0.211
Elevation (Feet)
1.70
1.60
1.50
1.40
As constructed
After stressing
FW Release
Best Fit
Design Final Elev.
Final Elev. (M/EI Camber)
Days After Falsework Release
MEYER ROAD UC Br # 20-242 Right (Right Side)

Figure 58



DEFLECTION AT ℄ OF SPAN
Span 1 Plan Camber = 0.17' Initial Elev. = 99.839 Span Length = 145.3'
Adj. M/EI Camber = 0.117
9.90
9.80
9.70
9.60
Elevation (Feet)
Best Fit
Final Elev. (M/EI Camber)
Design Final Elev.
As constructed
After stressing
FW Release
1 2 3 4 5 6 7 8 9 10 20 30 40 50 60 70 80 90 100 200 300 400 500 600 700 800 900 1000 2000 3000 4000 5000 6000 7000 8000 9000 10000
Days After Falsework Release
MEYER ROAD UC Br # 20-242 Left (Left Side)

Figure 59

DEFLECTION AT ℄ OF SPAN

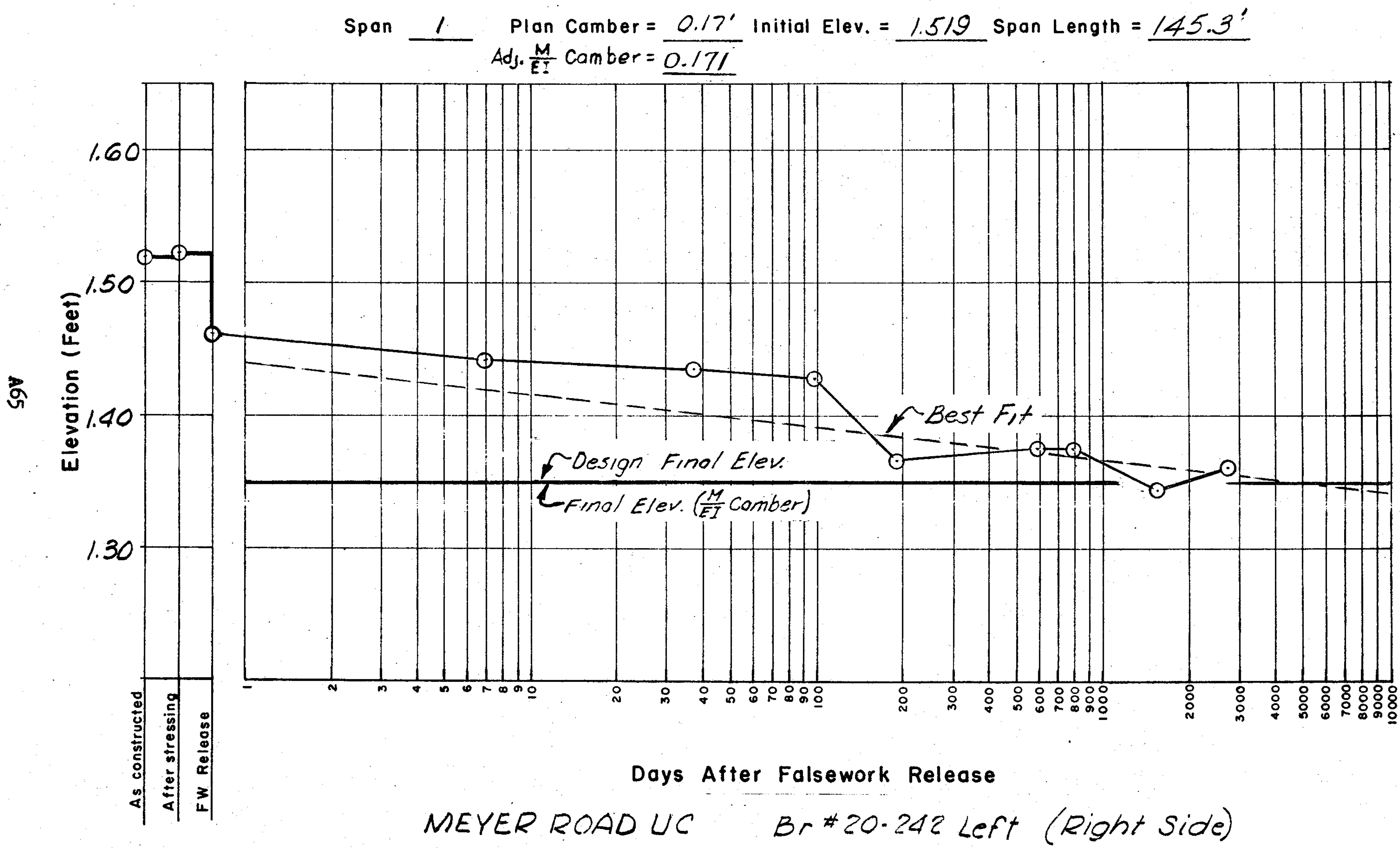


Figure 60

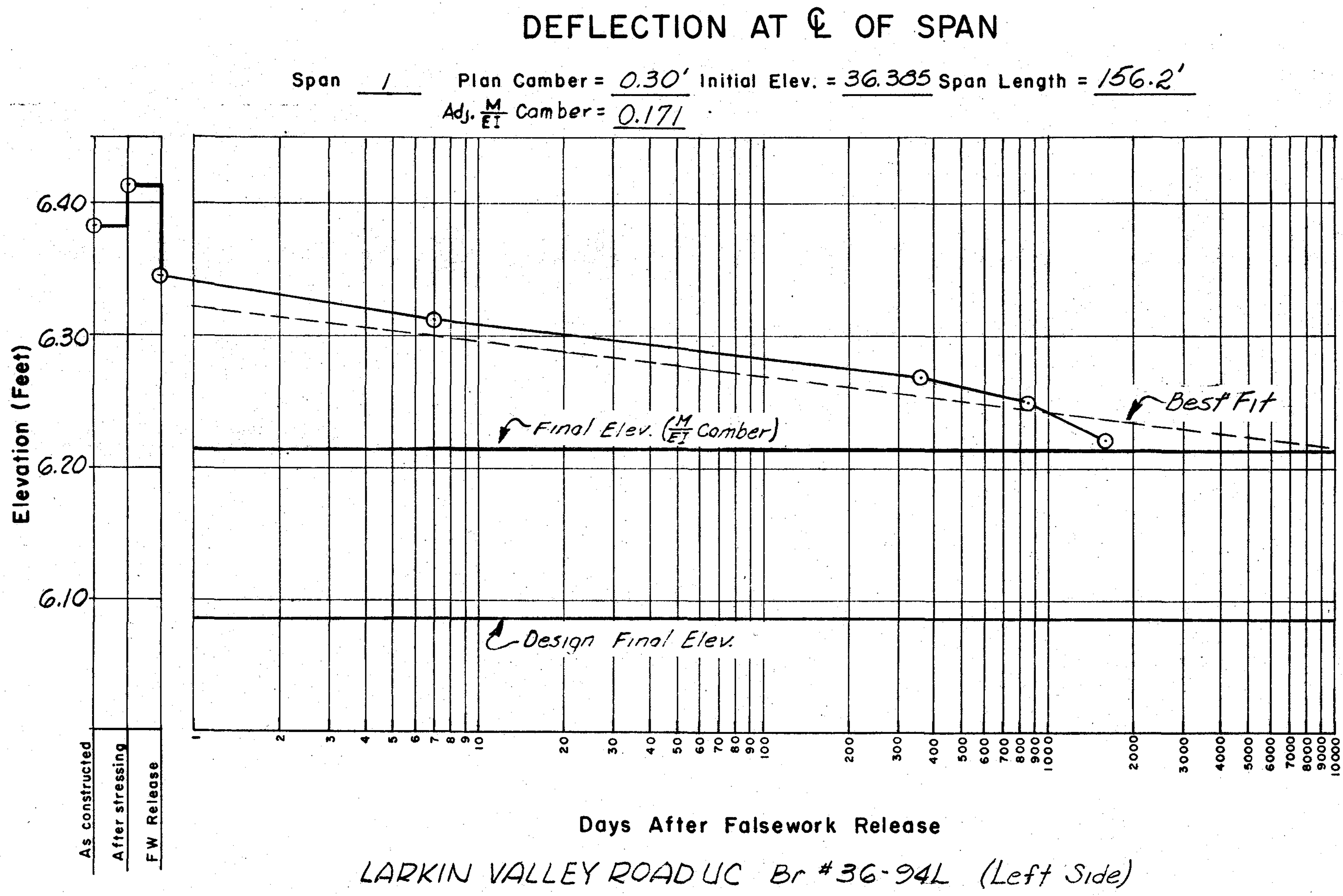
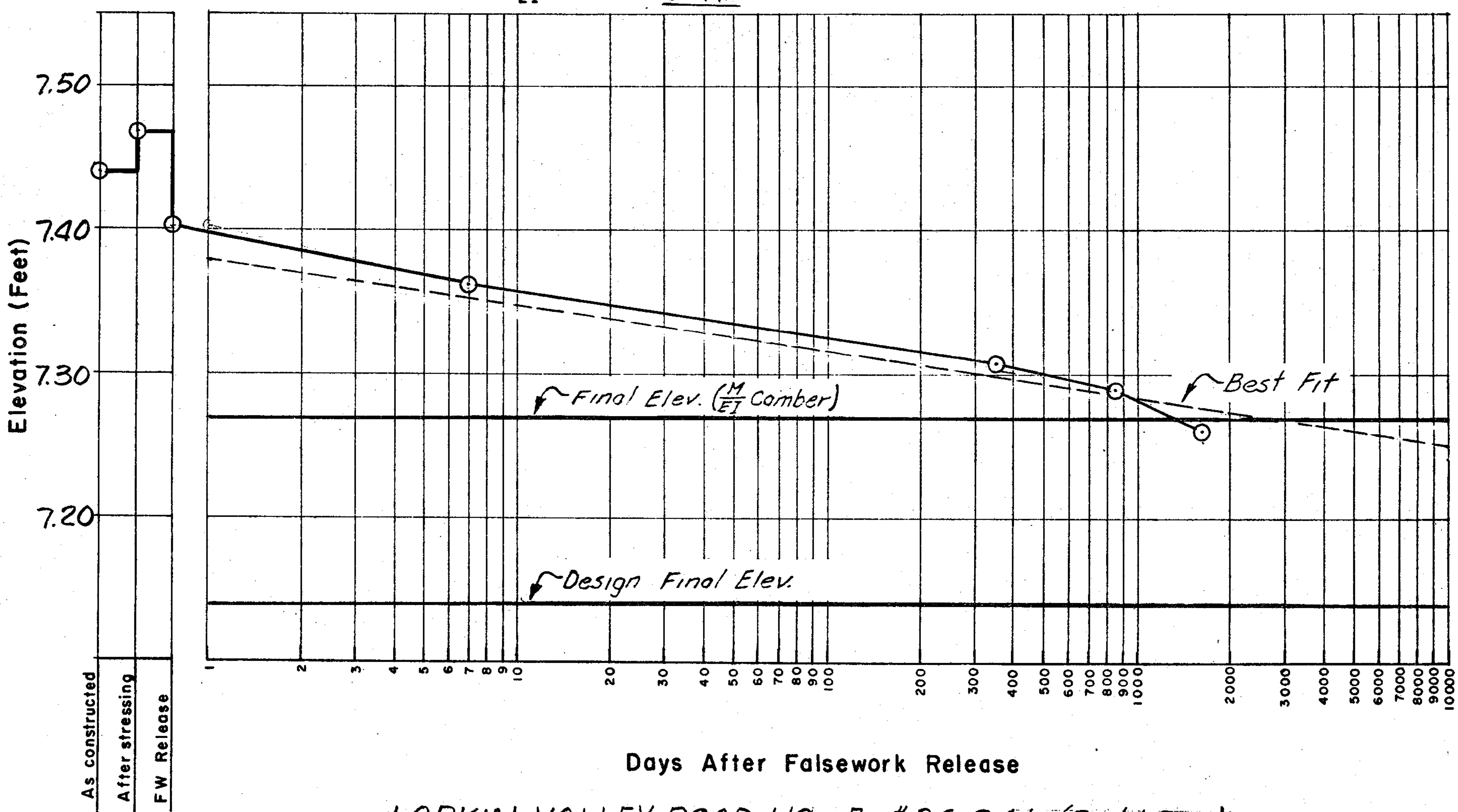

DEFLECTION AT ℄ OF SPAN
Span 1 Plan Camber = 0.30' Initial Elev. = 36.385 Span Length = 156.2'
Adj. M/EI Camber = 0.171
6.40
6.30
6.20
6.10
Elevation (Feet)
Best Fit
Final Elev. (M/EI Camber)
Design Final Elev.
As constructed
After stressing
FW Release
1
2
3
4
5
6
7
8
9
10
20
30
40
50
60
70
80
90
100
200
300
400
500
600
700
800
900
1000
2000
3000
4000
5000
6000
7000
8000
9000
10000
Days After Falsework Release
LARKIN VALLEY ROAD UC Br #36-94L (Left Side)

Figure 61

DEFLECTION AT ℄ OF SPAN

Span 1 Plan Camber = 0.30' Initial Elev. = 37.440 Span Length = 156.2'

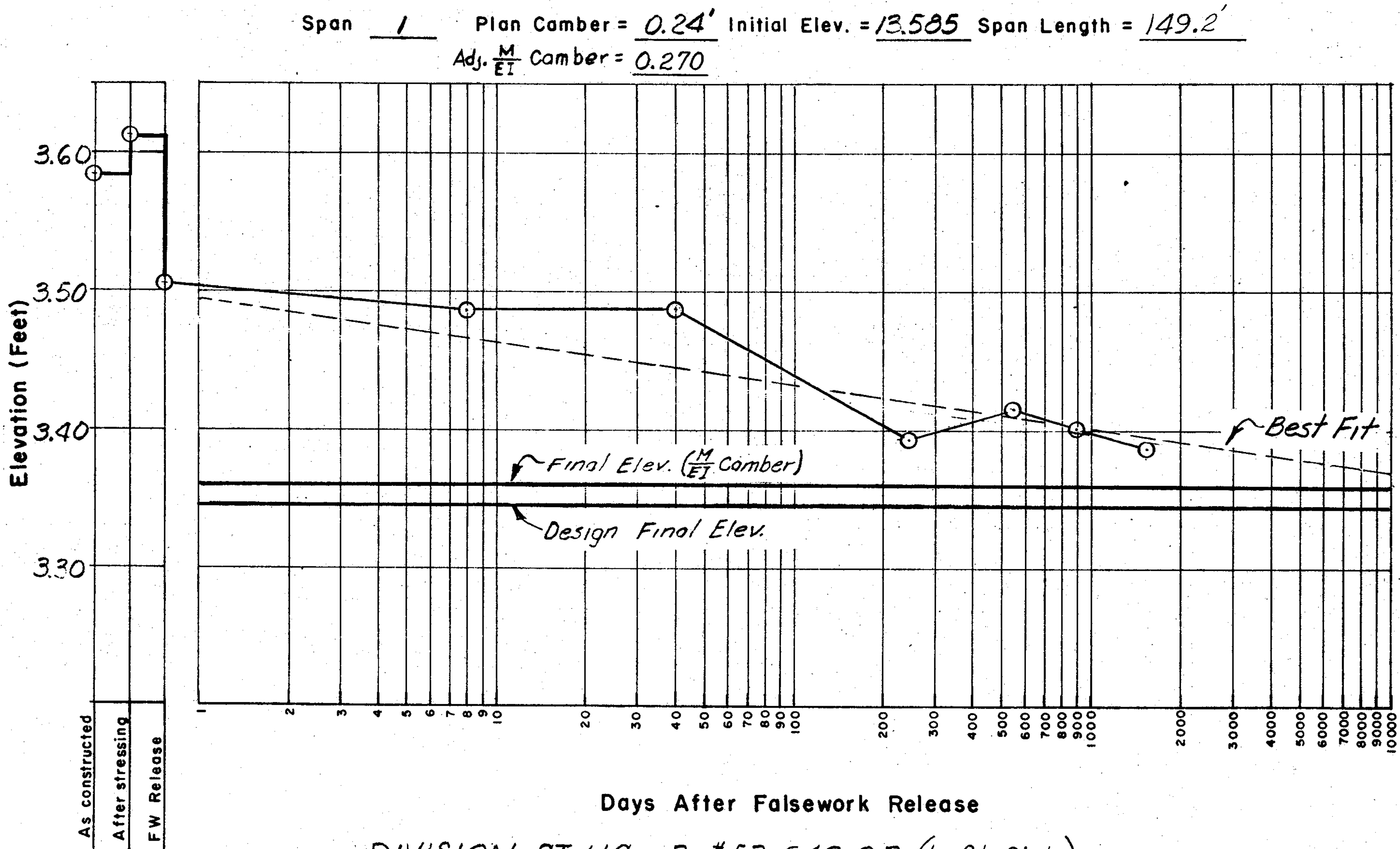
Adj. $\frac{M}{EI}$ Camber = 0.171



LARKIN VALLEY ROAD UC Br #36-94L (Right Side)

Figure 62

DEFLECTION AT ℄ OF SPAN



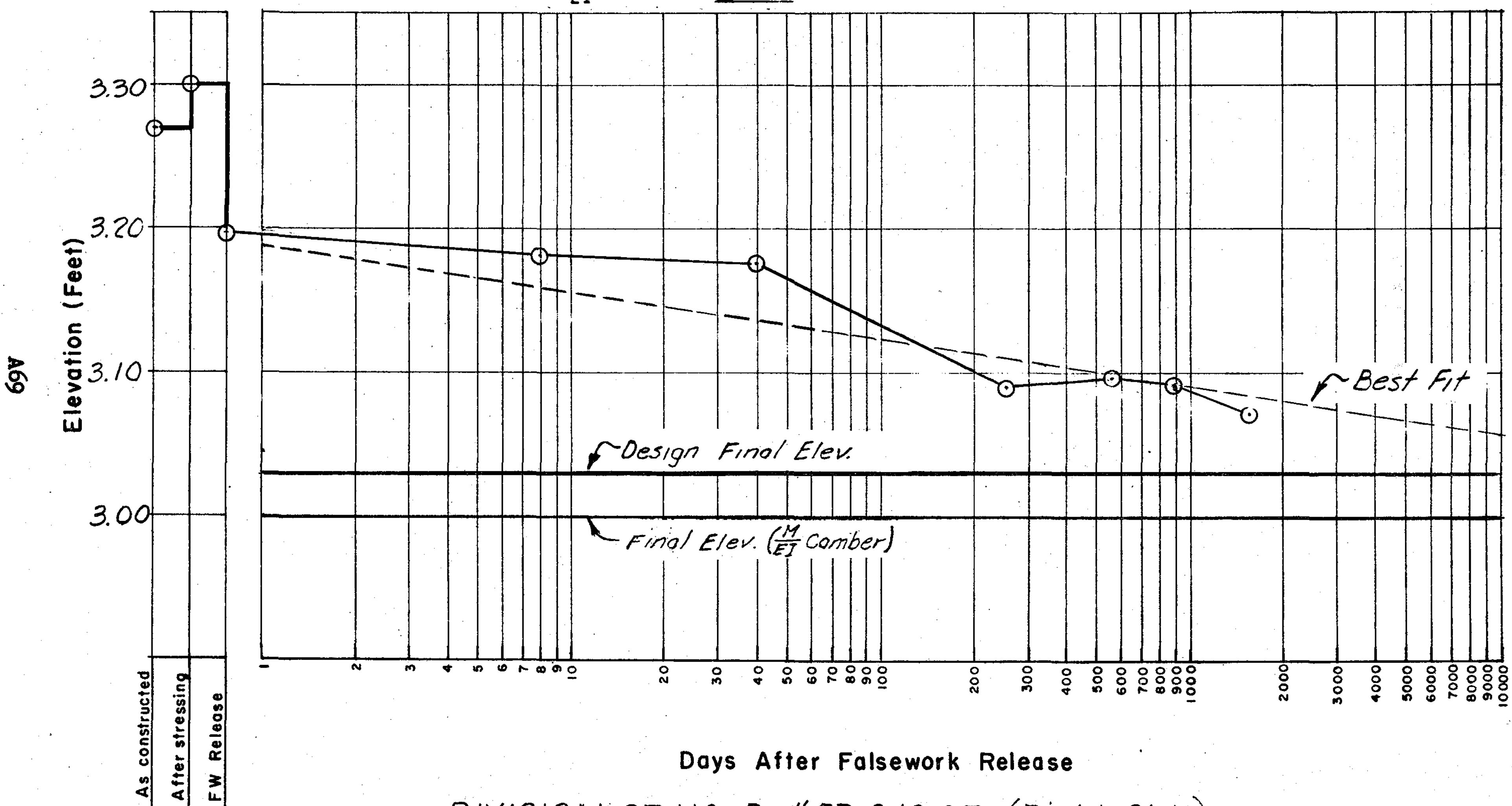
DIVISION ST UC Br #57-646 QR (Left Side)

Figure 63

DEFLECTION AT ℄ OF SPAN

Span 1 Plan Camber = 0.24' Initial Elev. = 13.27 Span Length = 149.2'

Adj. $\frac{M}{EI}$ Camber = 0.270



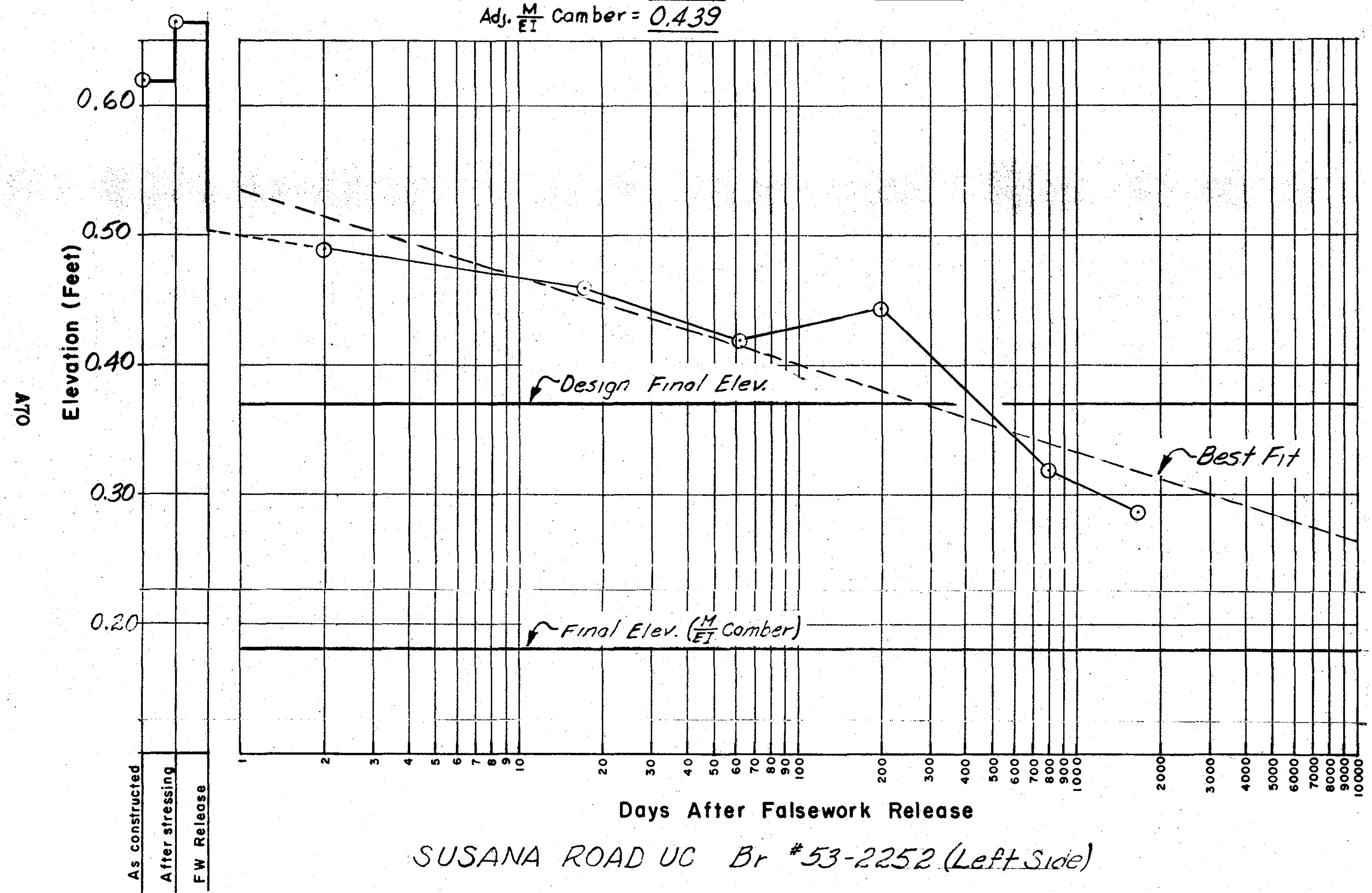
DIVISION ST UC Br #57-646 QR (Right Side)

Figure 64

DEFLECTION AT ℄ OF SPAN

Span 1 Plan Camber = 0.25 Initial Elev. = 10.62 Span Length = 168.4'

Adj. $\frac{M}{EI}$ Camber = 0.439



SUSANA ROAD UC Br #53-2252 (Left Side)

Figure 65

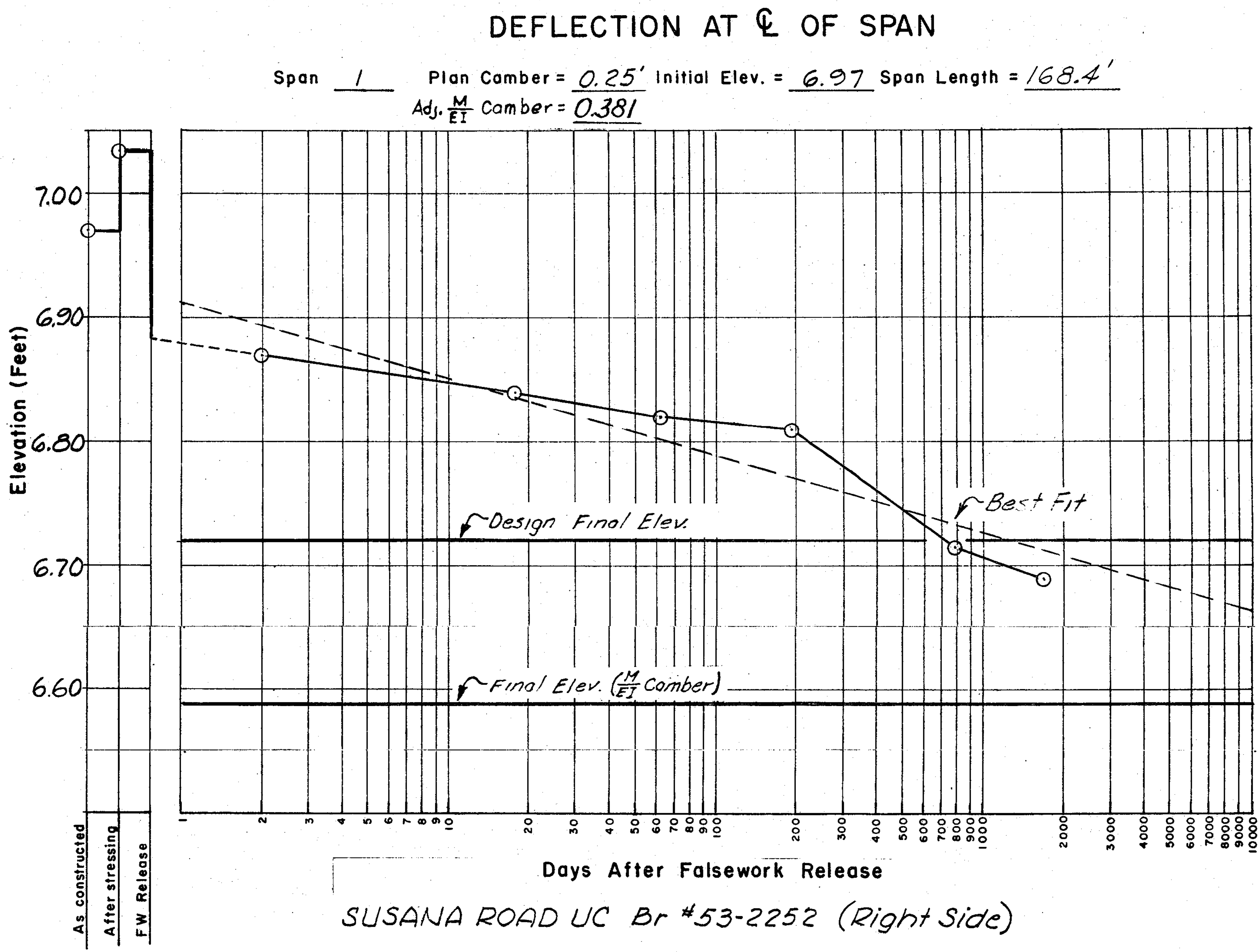

DEFLECTION AT ℄ OF SPAN
Span 1 Plan Camber = 0.25' Initial Elev. = 6.97 Span Length = 168.4'
Adj. M/EI Camber = 0.381
Elevation (Feet)
7.00
6.90
6.80
6.70
6.60
ATL
Design Final Elev.
Final Elev. (M/EI Camber)
Best Fit
As constructed
After stressing
FW Release
1 2 3 4 5 6 7 8 9 10 20 30 40 50 60 70 80 90 100 200 300 400 500 600 700 800 900 1000 2000 3000 4000 5000 6000 7000 8000 9000 10000
Days After Falsework Release
SUSANA ROAD UC Br #53-2252 (Right Side)

Figure 66

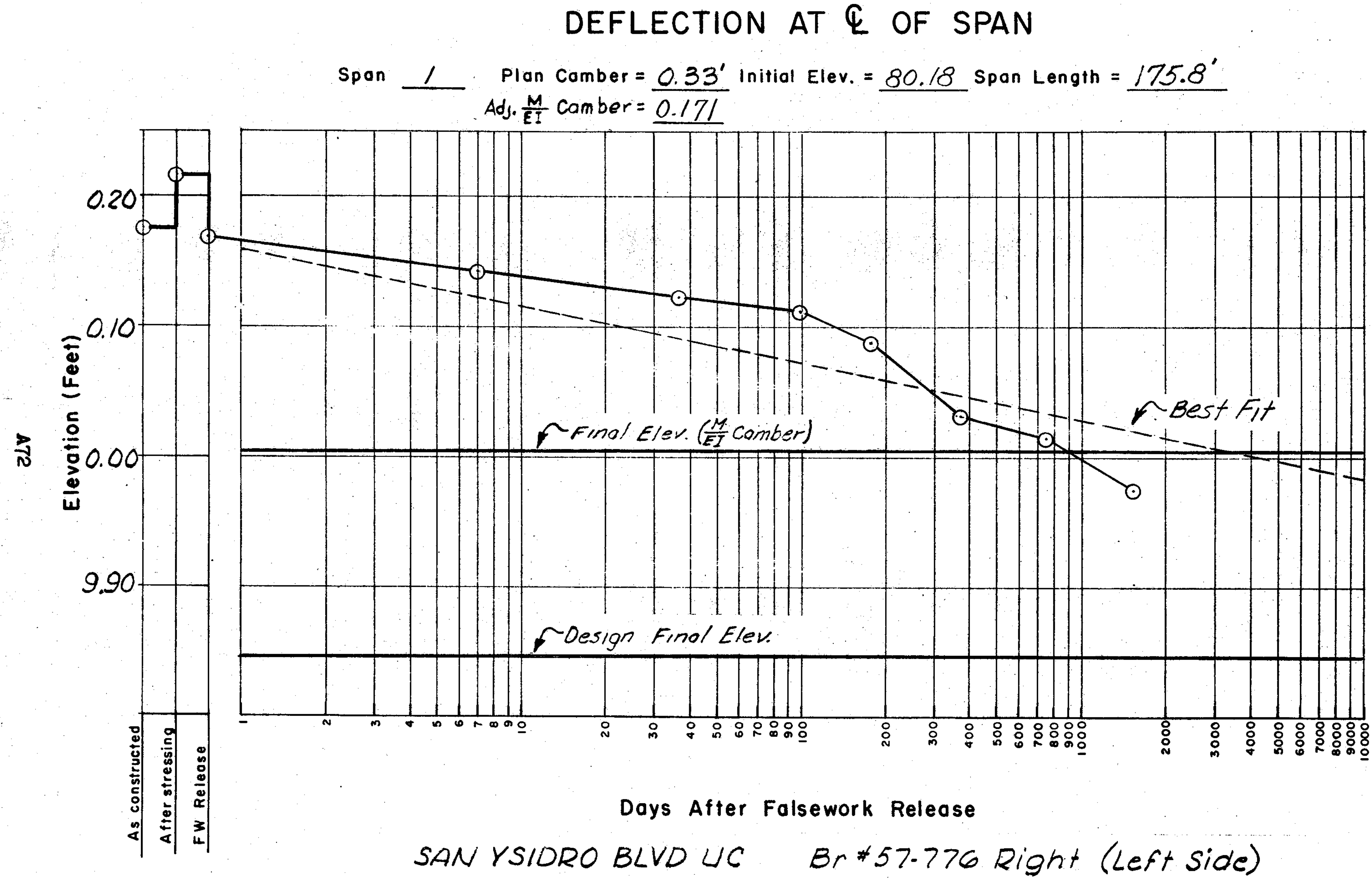
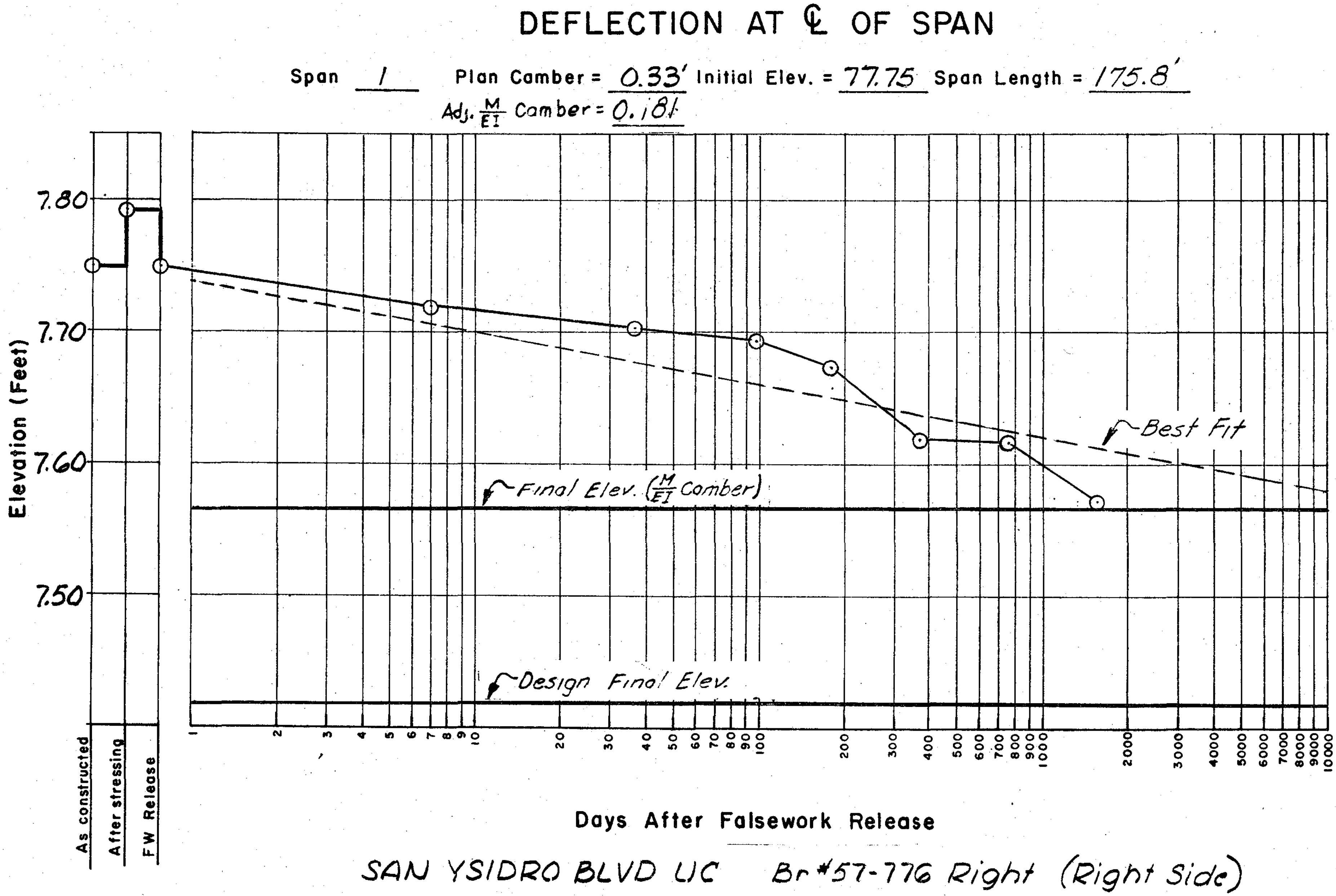

DEFLECTION AT ℄ OF SPAN
Span 1 Plan Camber = 0.33' Initial Elev. = 80.18 Span Length = 175.8'
Adj. M/EI Camber = 0.171
0.20
0.10
0.00
9.90
Elevation (Feet)
Final Elev. (M/EI Camber)
Best Fit
Design Final Elev.
As constructed
After stressing
FW Release
1
2
3
4
5
6
7
8
9
10
20
30
40
50
60
70
80
90
100
200
300
400
500
600
700
800
900
1000
2000
3000
4000
5000
6000
7000
8000
9000
10000
Days After Falsework Release
SAN YSIDRO BLVD UC Br #57-776 Right (Left Side)

Figure 67



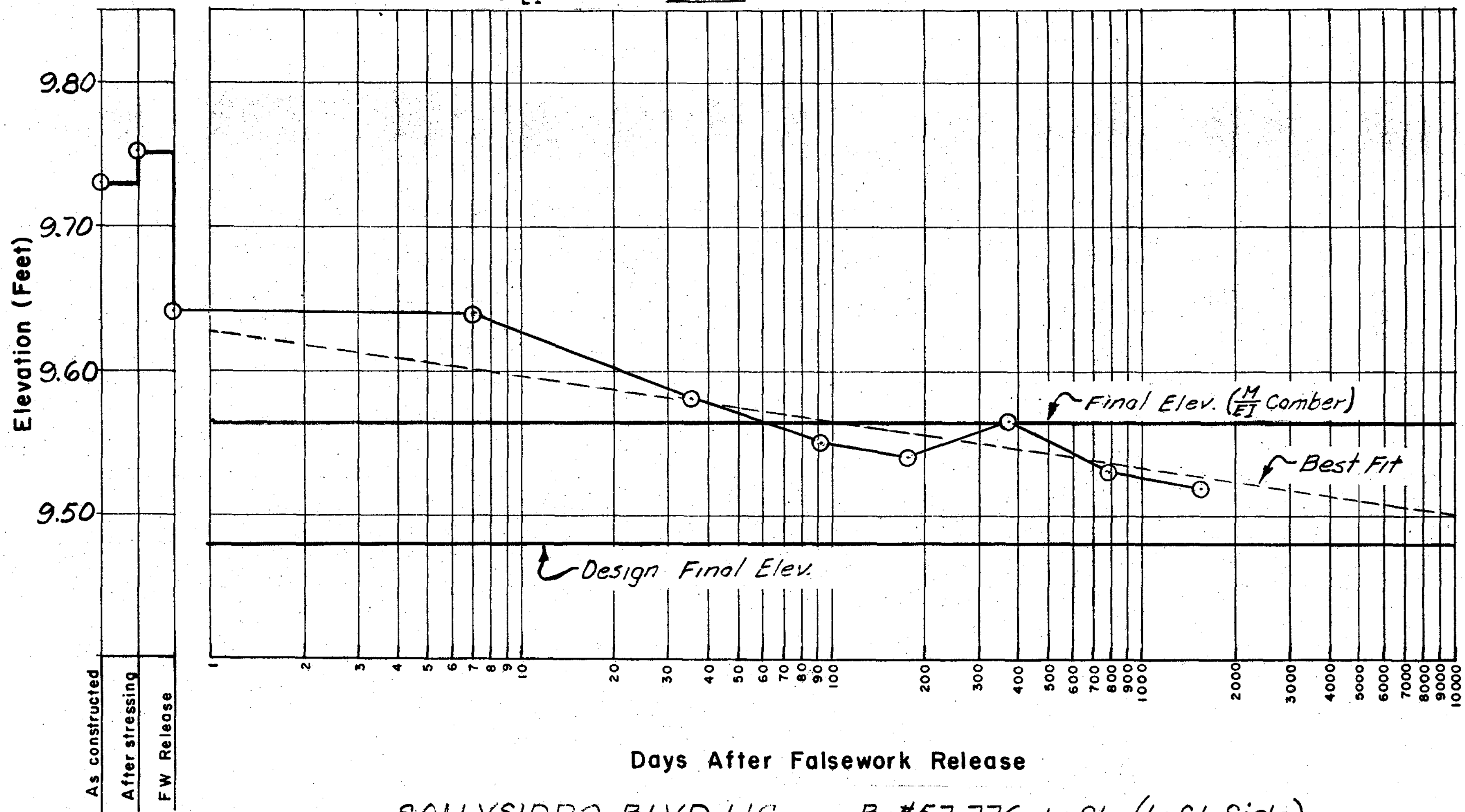
A73

Figure 68

DEFLECTION AT ℄ OF SPAN

Span 1 Plan Camber = 0.25' Initial Elev. = 79.73 Span Length = 167.5'

Adj. $\frac{M}{EI}$ Camber = 0.164



SAN YSIDRO BLVD UC Br#57-776 Left (Left Side)

Figure 69

A74

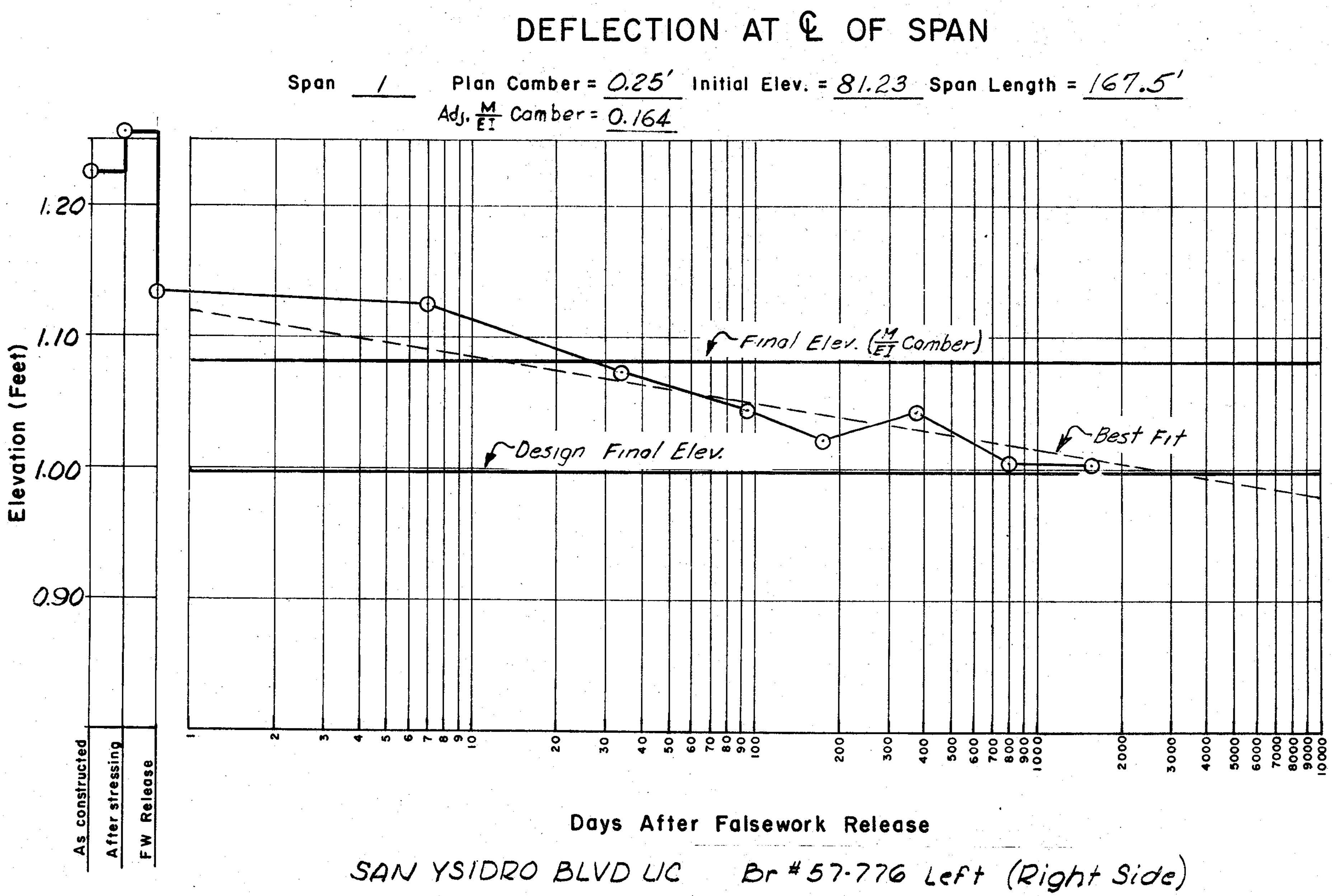

DEFLECTION AT ℄ OF SPAN
Span 1 Plan Camber = 0.25' Initial Elev. = 81.23 Span Length = 167.5'
Adj. M/EI Camber = 0.164
Elevation (Feet)
1.20
1.10
1.00
0.90
Final Elev. (M/EI Camber)
Design Final Elev.
Best Fit
As constructed
After stressing
FW Release
1
2
3
4
5
6
7
8
9
10
20
30
40
50
60
70
80
90
100
200
300
400
500
600
700
800
900
1000
2000
3000
4000
5000
6000
7000
8000
9000
10000
Days After Falsework Release
SAN YSIDRO BLVD UC Br # 57-776 Left (Right Side)

Figure 70

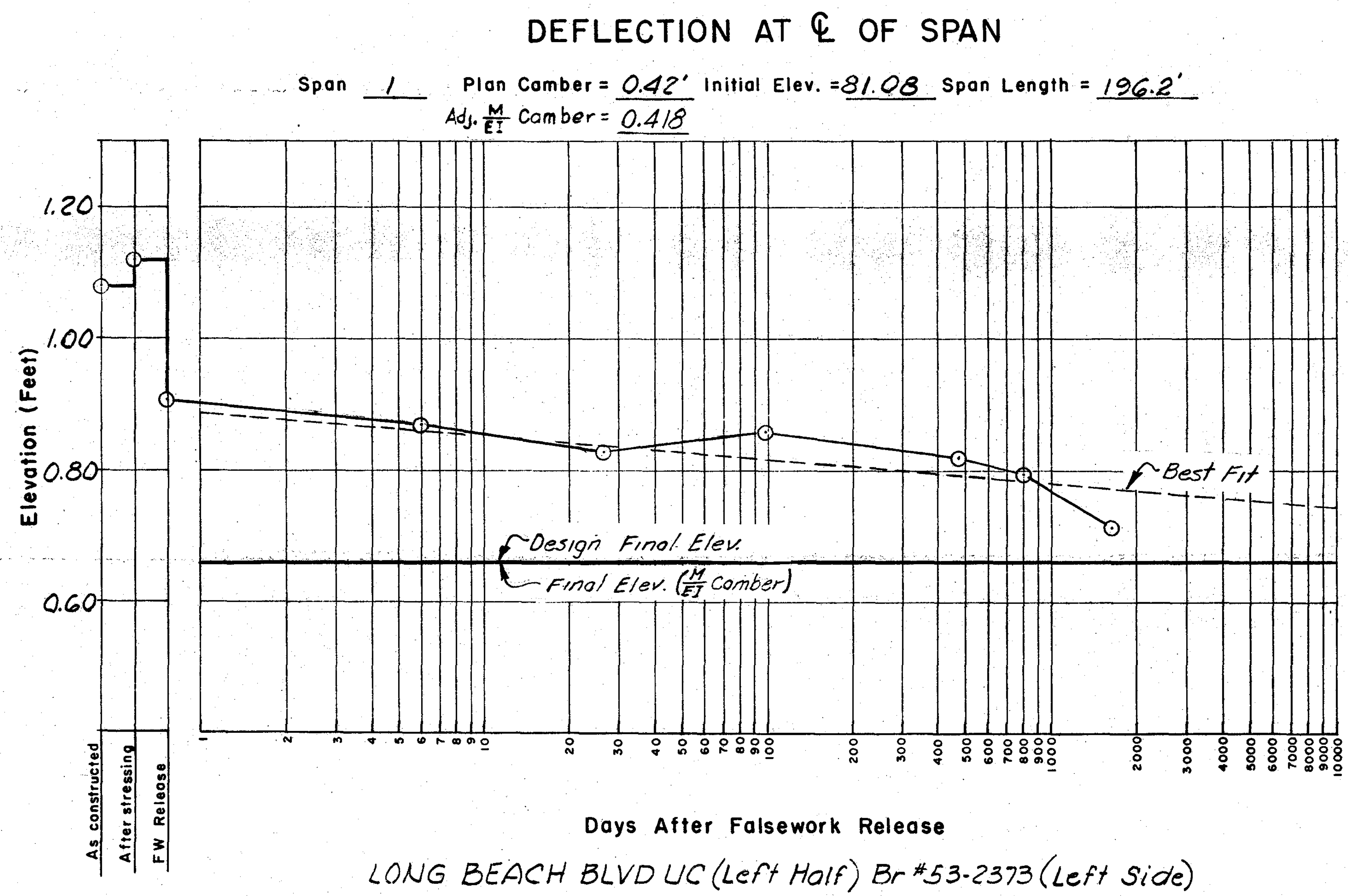
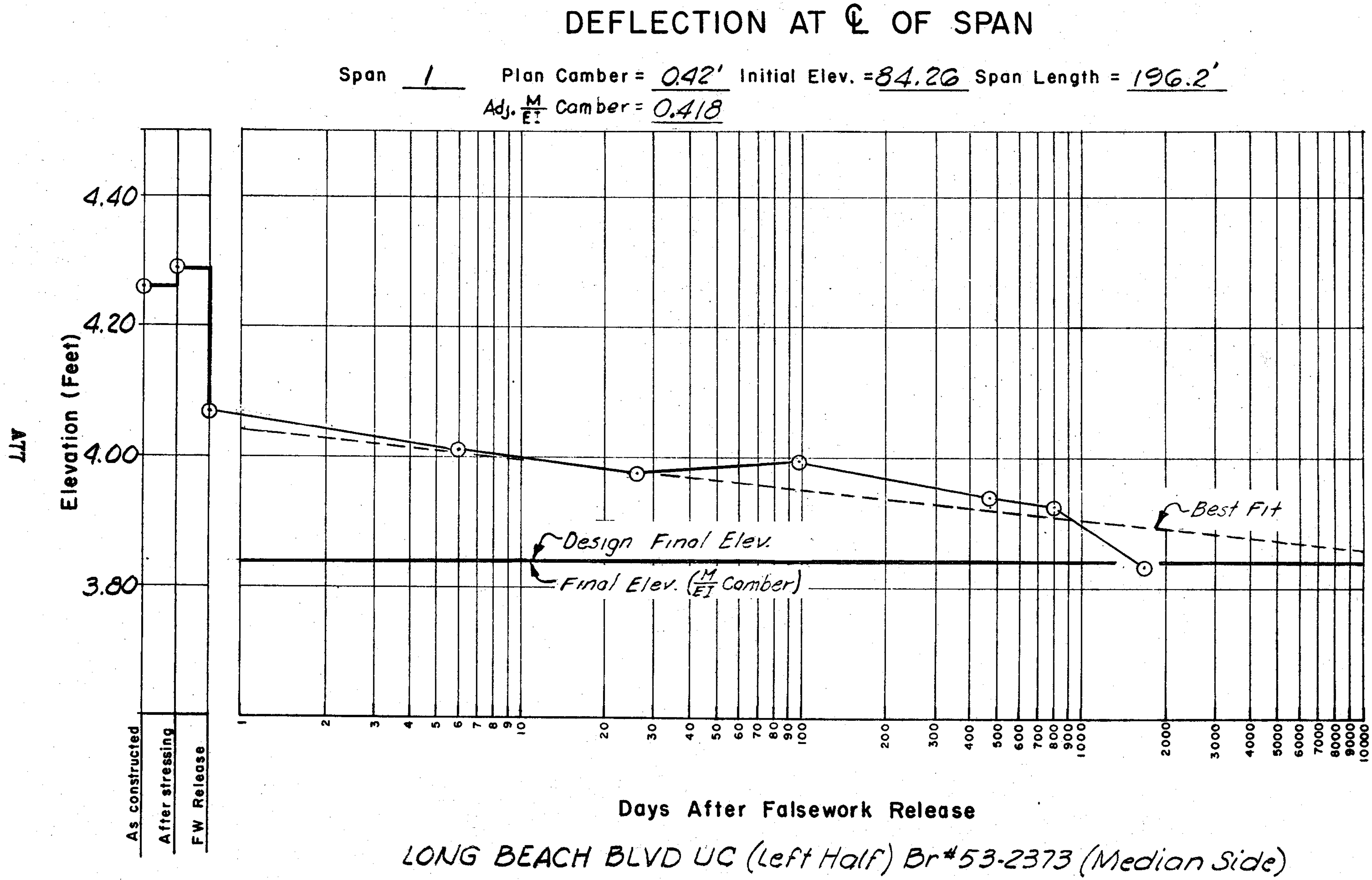

DEFLECTION AT ℄ OF SPAN
Span 1
Plan Camber = 0.42'
Initial Elev. = 81.08
Span Length = 196.2'
Adj. M/EI Camber = 0.418
1.20
1.00
0.80
0.60
Elevation (Feet)
Best Fit
Design Final Elev.
Final Elev. (M/EI Camber)
As constructed
After stressing
FW Release
1
2
3
4
5
6
7
8
9
10
20
30
40
50
60
70
80
90
100
200
300
400
500
600
700
800
900
1000
2000
3000
4000
5000
6000
7000
8000
9000
10000
Days After Falsework Release
LONG BEACH BLVD UC (Left Half) Br #53-2373 (Left Side)

Figure 71



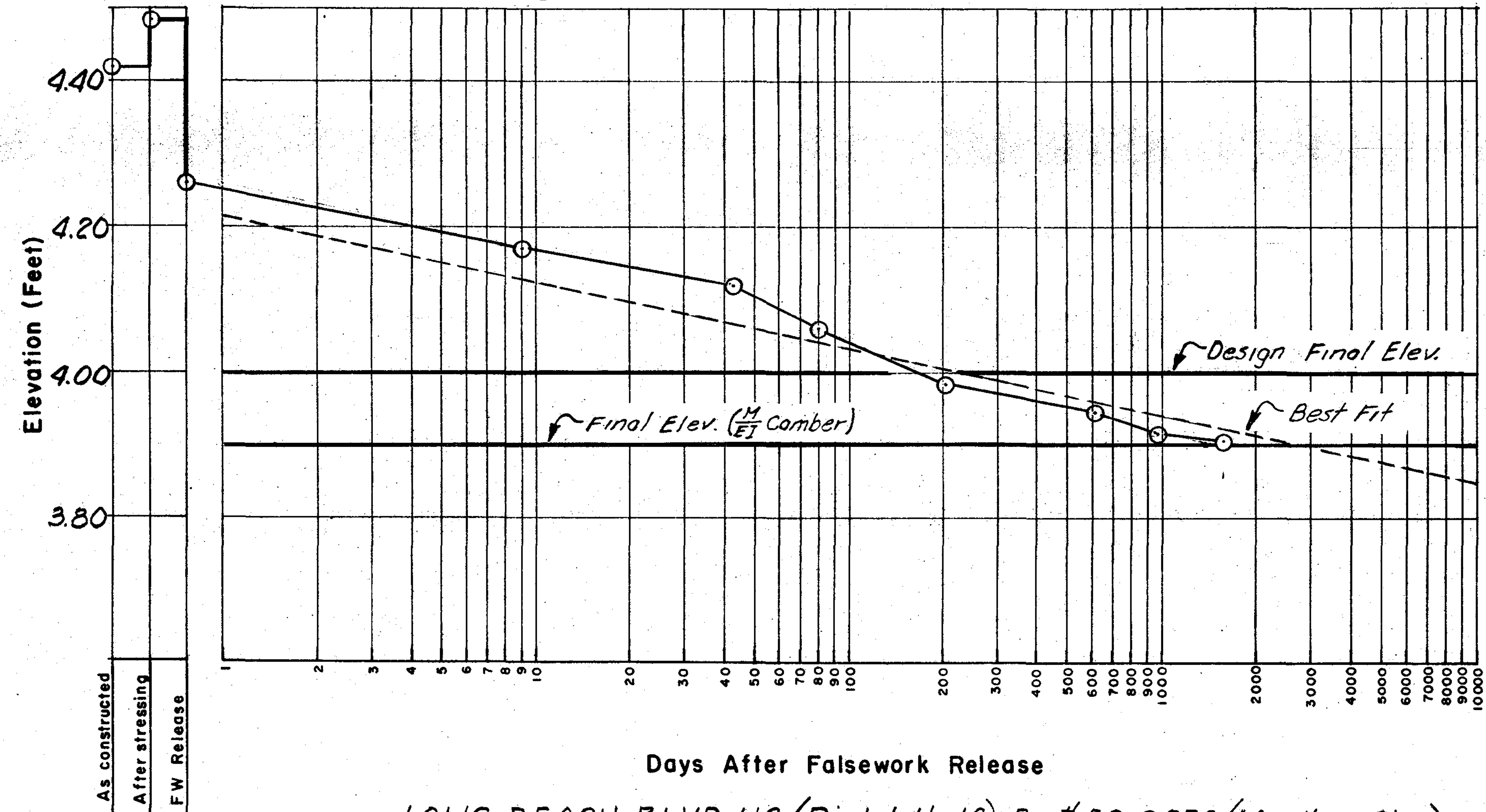
DEFLECTION AT ℄ OF SPAN
Span 1 Plan Camber = 0.42' Initial Elev. = 84.26 Span Length = 196.2'
Adj. M/EI Camber = 0.418
Elevation (Feet)
ATT
4.40
4.20
4.00
3.80
As constructed
After stressing
FW Release
1 2 3 4 5 6 7 8 9 10 20 30 40 50 60 70 80 90 100 200 300 400 500 600 700 800 900 1000 2000 3000 4000 5000 6000 7000 8000 9000 10000
Best Fit
Design Final Elev.
Final Elev. (M/EI Camber)
Days After Falsework Release
LONG BEACH BLVD UC (Left Half) Br#53-2373 (Median Side)

Figure 72

DEFLECTION AT ℄ OF SPAN

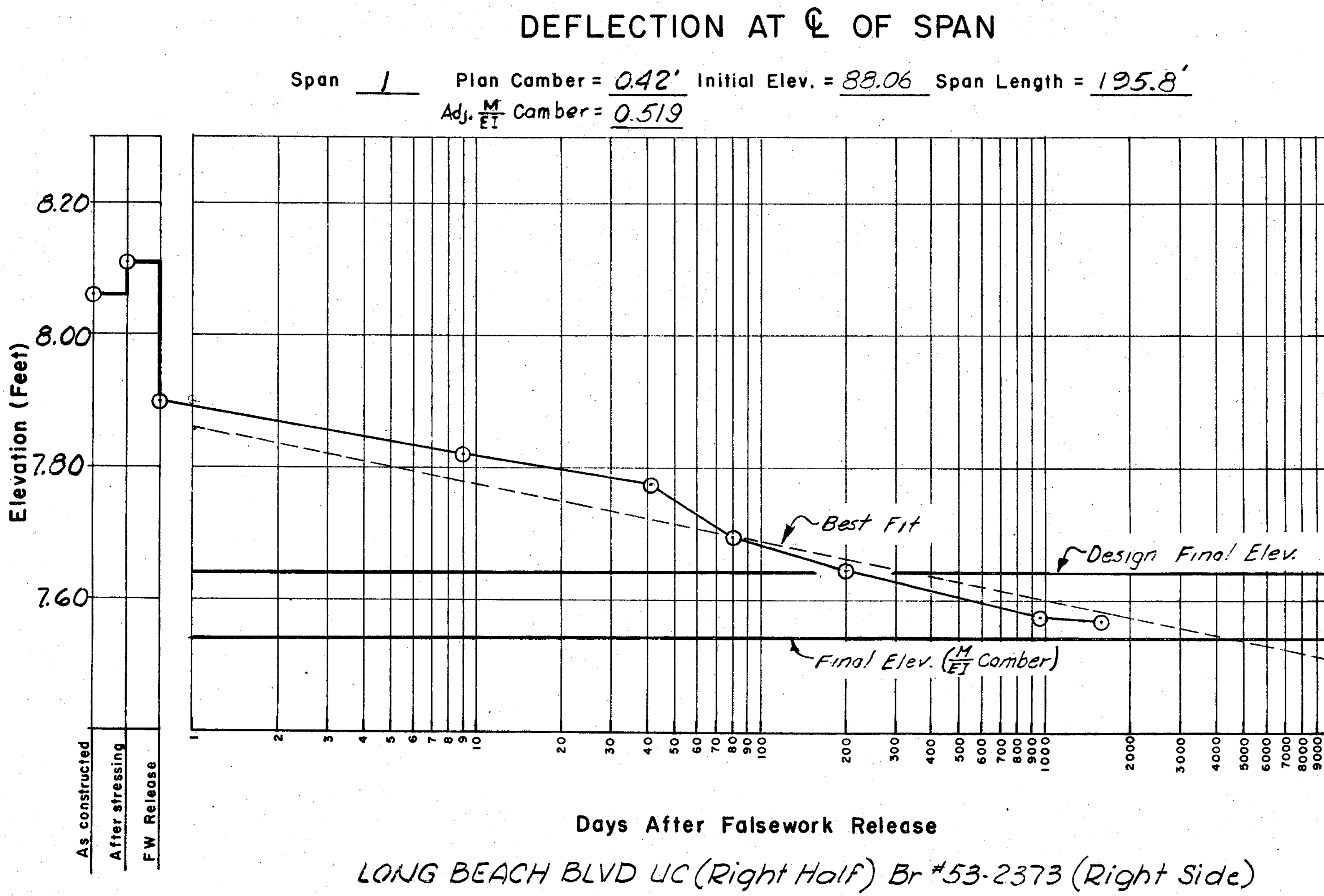
Span 1 Plan Camber = 0.42' Initial Elev. = 84.42 Span Length = 195.8'

Adj. $\frac{M}{EI}$ Camber = 0.519



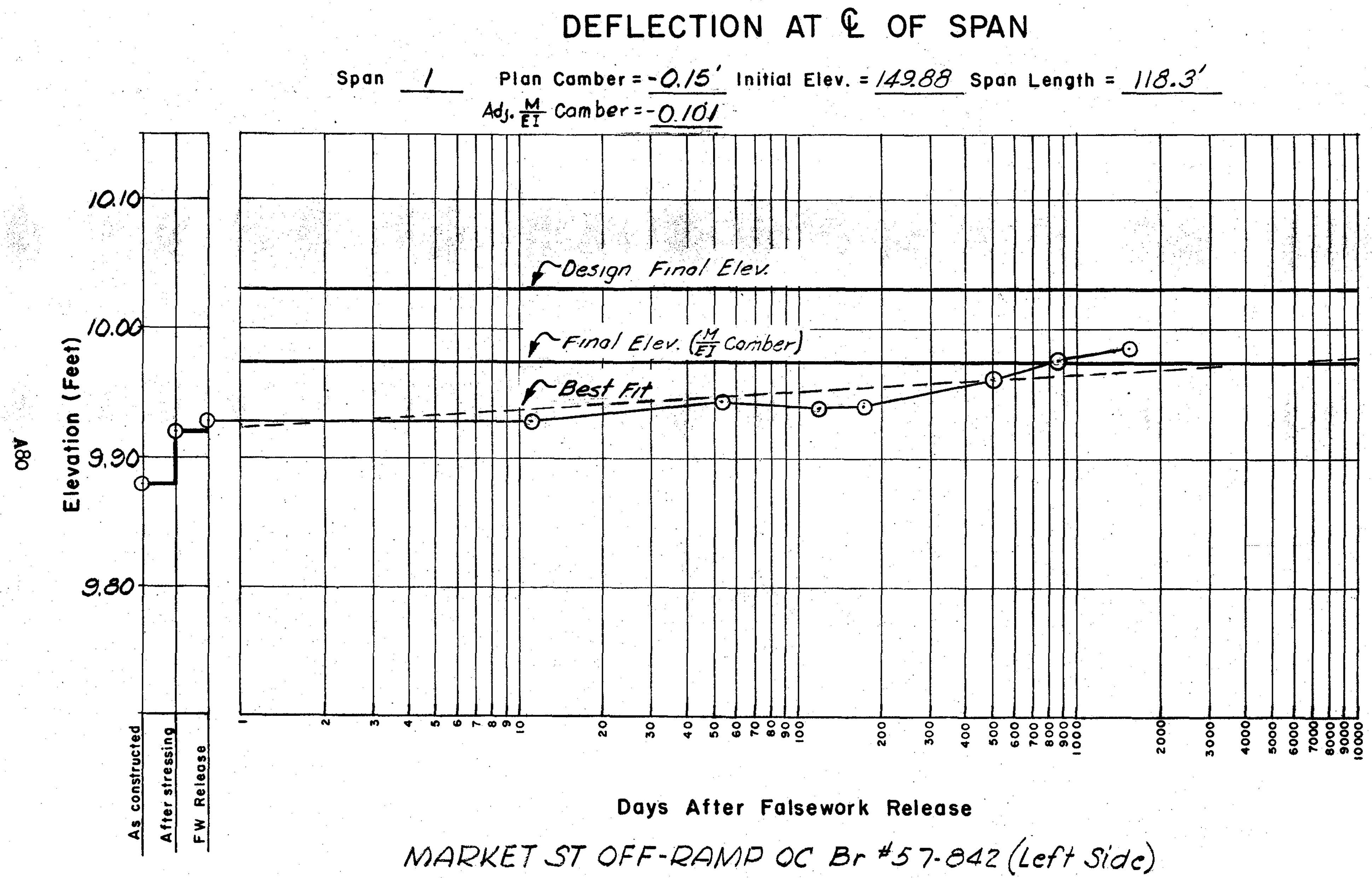
LONG BEACH BLVD UC (Right Half) Br #53-2373 (Median Side)

Figure 73



DEFLECTION AT ℄ OF SPAN
Span 1 Plan Camber = 0.42' Initial Elev. = 88.06 Span Length = 195.8'
Adj. M/EI Camber = 0.519
8.20
8.00
7.80
7.60
Elevation (Feet)
Best Fit
Design Final Elev.
Final Elev. (M/EI Camber)
As constructed
After stressing
FW Release
1
2
3
4
5
6
7
8
9
10
20
30
40
50
60
70
80
90
100
200
300
400
500
600
700
800
900
1000
2000
3000
4000
5000
6000
7000
8000
9000
10000
Days After Falsework Release
LONG BEACH BLVD UC (Right Half) Br #53-2373 (Right Side)

Figure 74



DEFLECTION AT ℄ OF SPAN
Span 1 Plan Camber = -0.15' Initial Elev. = 149.88 Span Length = 118.3'
Adj. M/EI Camber = -0.101
Design Final Elev.
Final Elev. (M/EI Camber)
Best Fit
Elevation (Feet)
10.10
10.00
9.90
9.80
As constructed
After stressing
FW Release
Days After Falsework Release
MARKET ST OFF-RAMP OC Br #57-842 (Left Side)

Figure 75

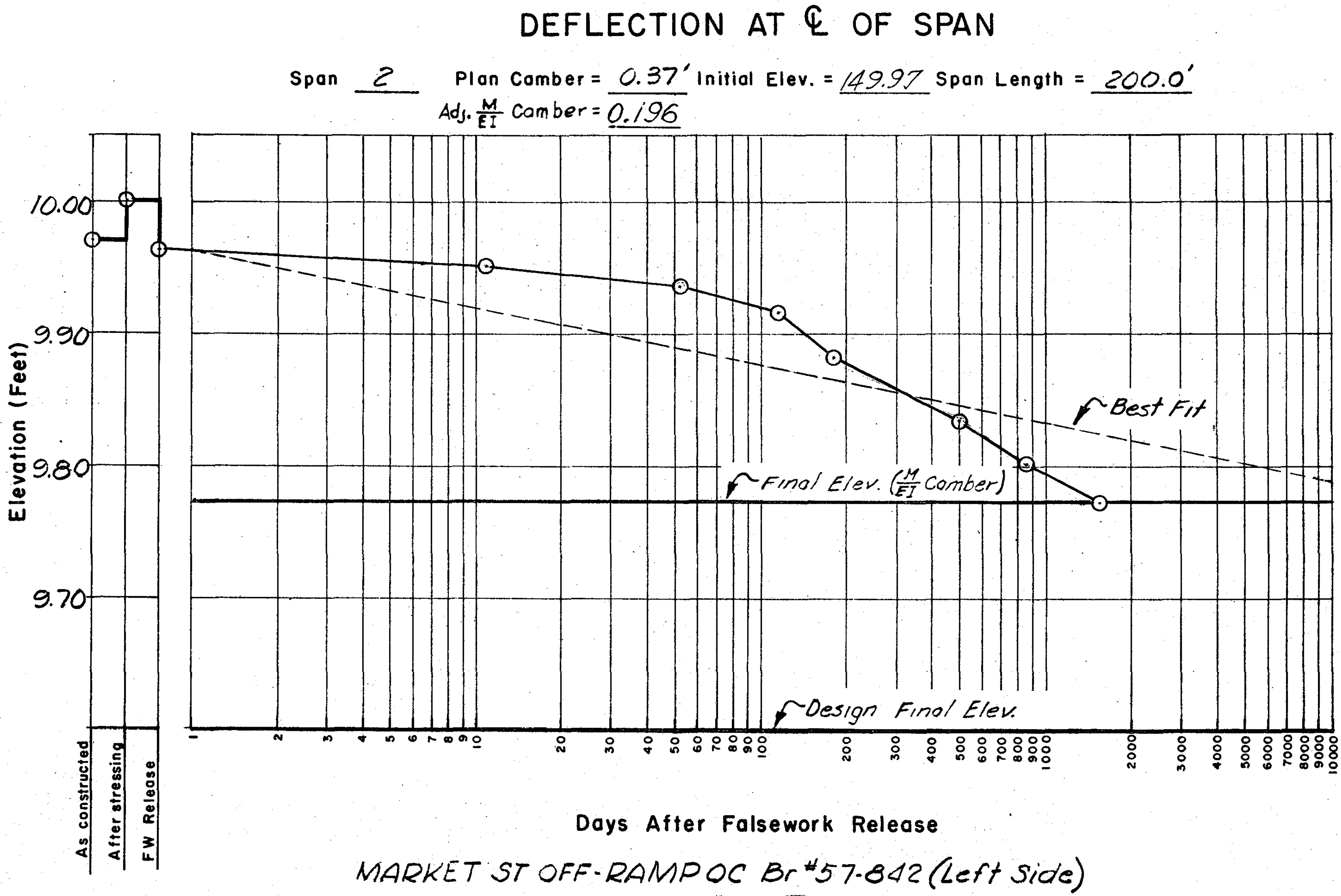
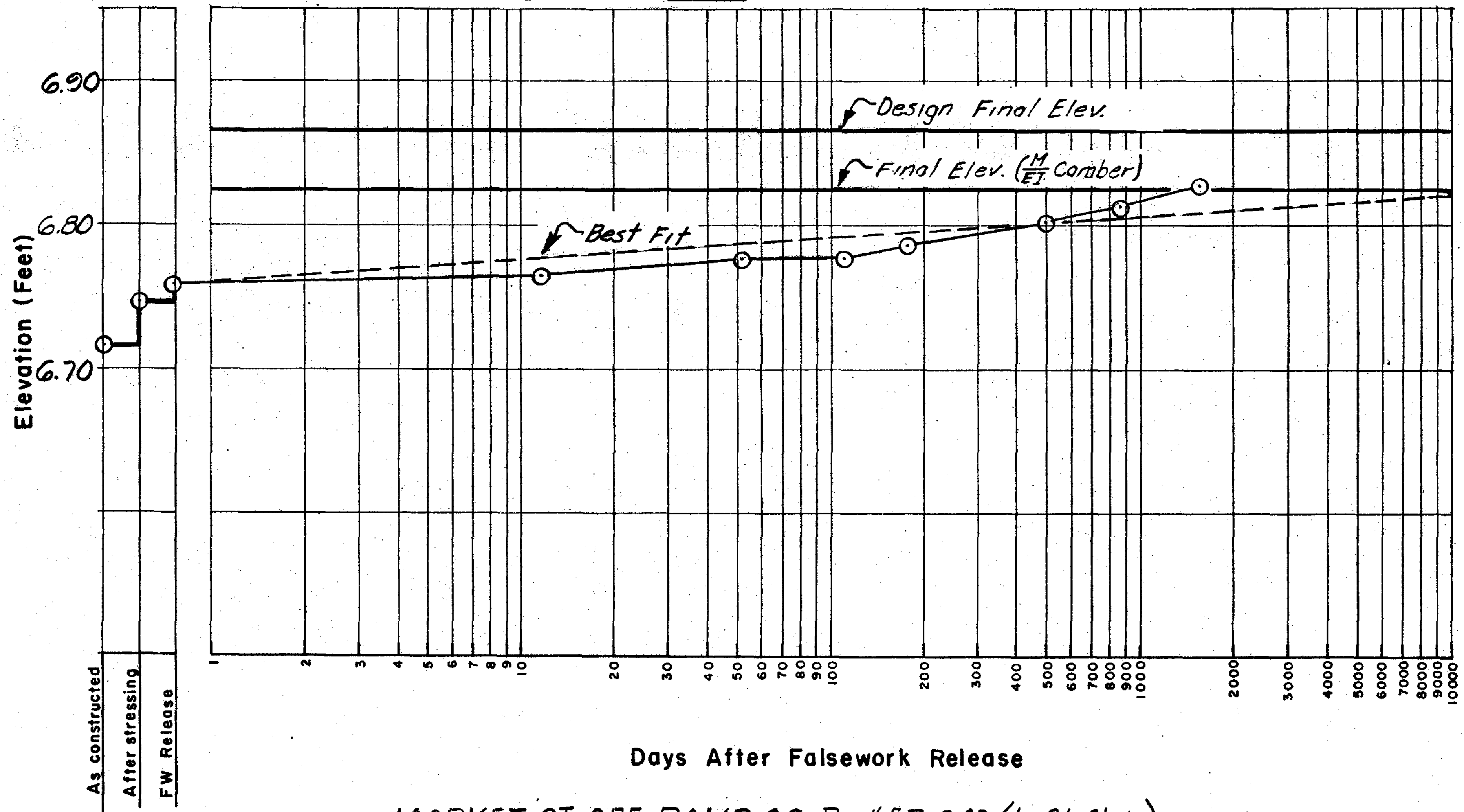

DEFLECTION AT ℄ OF SPAN
Span 2 Plan Camber = 0.37' Initial Elev. = 149.97 Span Length = 200.0'
Adj. M/EI Camber = 0.196
10.00
9.90
9.80
9.70
Elevation (Feet)
Best Fit
Final Elev. (M/EI Camber)
Design Final Elev.
As constructed
After stressing
FW Release
1 2 3 4 5 6 7 8 9 10 20 30 40 50 60 70 80 90 100 200 300 400 500 600 700 800 900 1000 2000 3000 4000 5000 6000 7000 8000 9000 10000
Days After Falsework Release
MARKET ST OFF-RAMP OC Br #57-842 (Left Side)

Figure 76

DEFLECTION AT ℄ OF SPAN

Span 3 Plan Camber = -0.15' Initial Elev. = 146.72 Span Length = 118.3'
Adj. $\frac{M}{EI}$ Camber = -.108



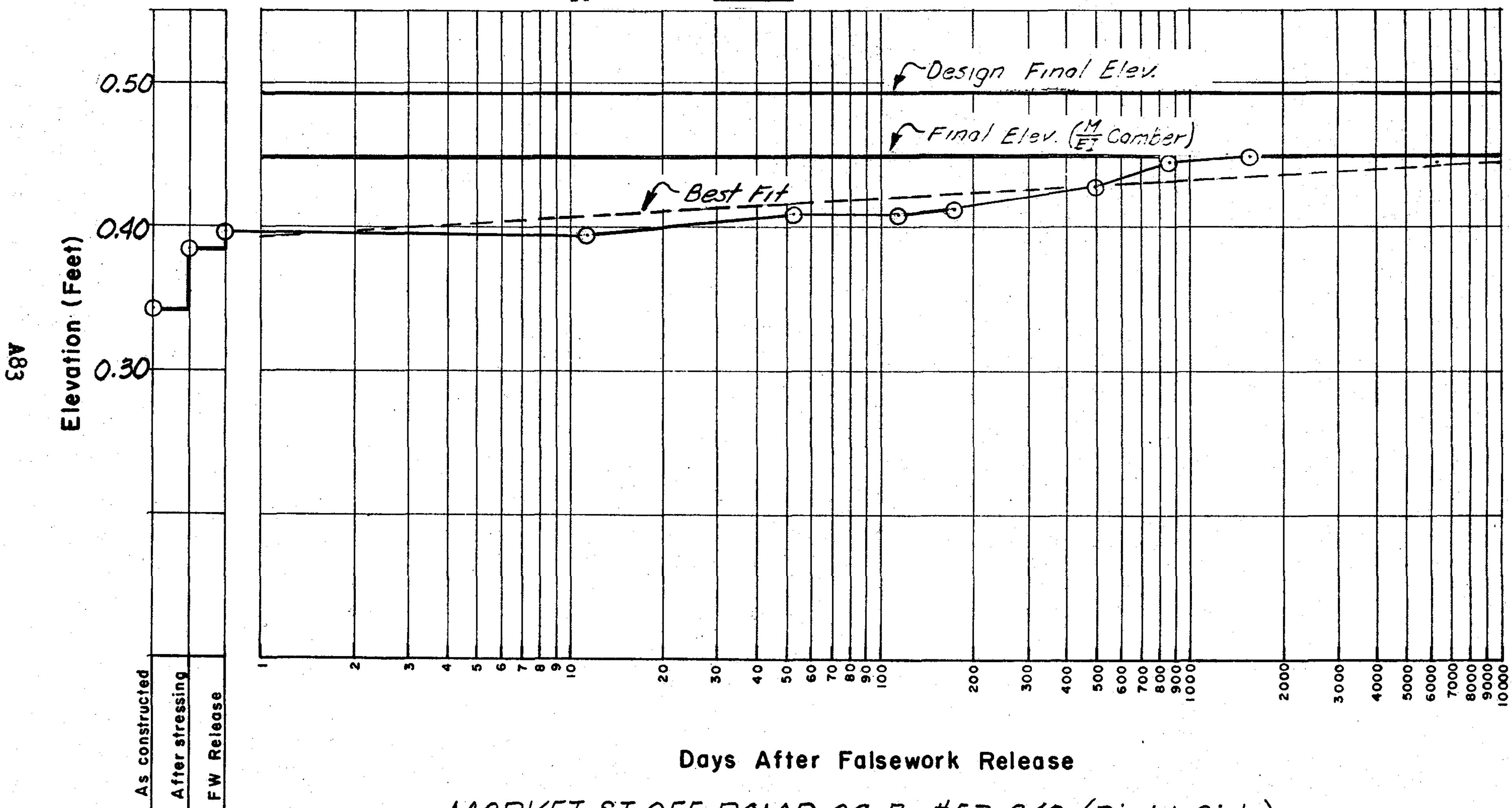
MARKET ST OFF-RAMP OC Br #57-842 (Left Side)

Figure 77

DEFLECTION AT ℄ OF SPAN

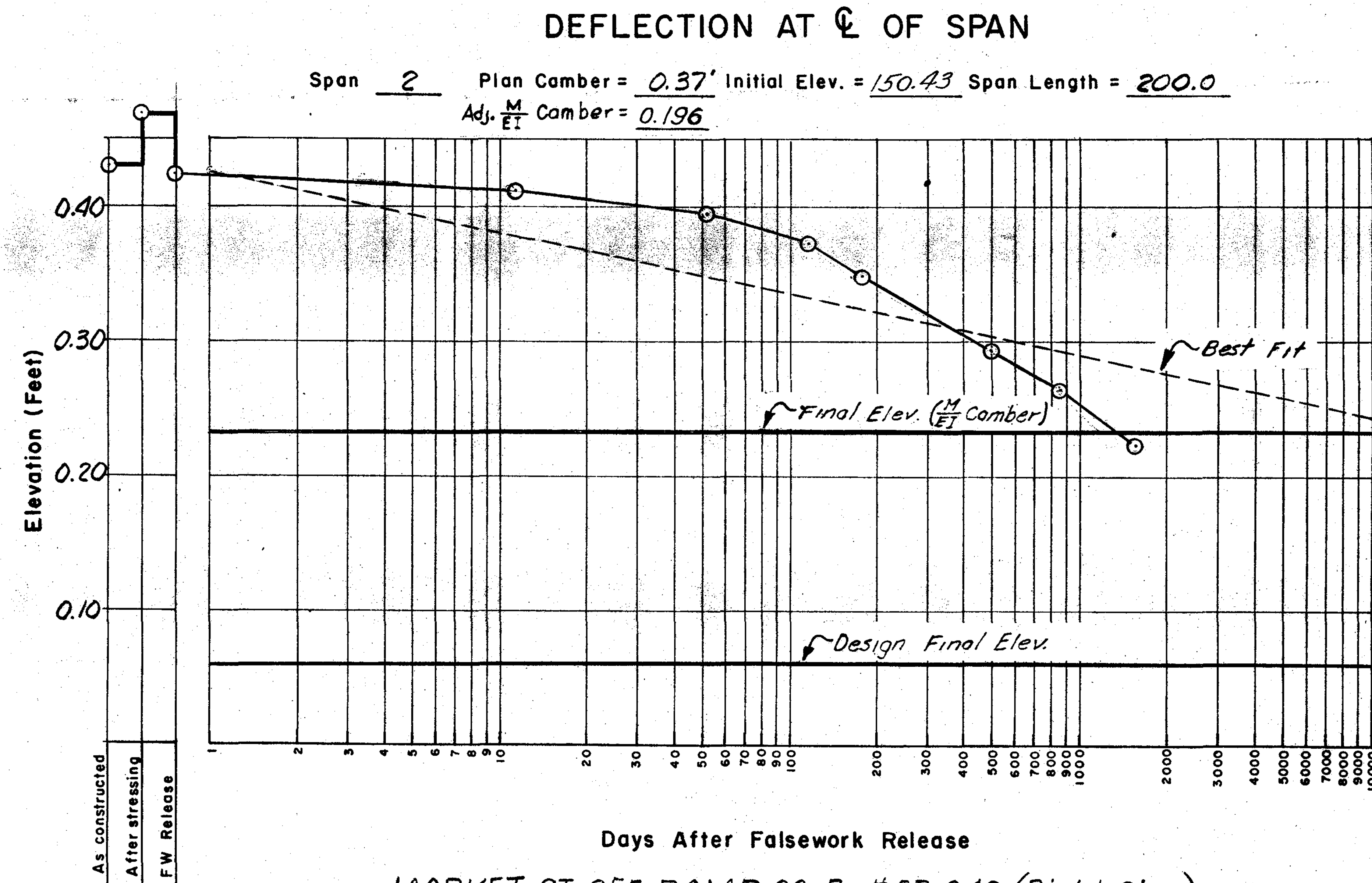
Span _1_ Plan Camber = _-0.15'_ Initial Elev. = _150.34_ Span Length = _118.3'_

Adj. $\frac{M}{EI}$ Camber = _-0.105_



MARKET ST OFF-RAMP OC Br #57-842 (Right Side)

Figure 78



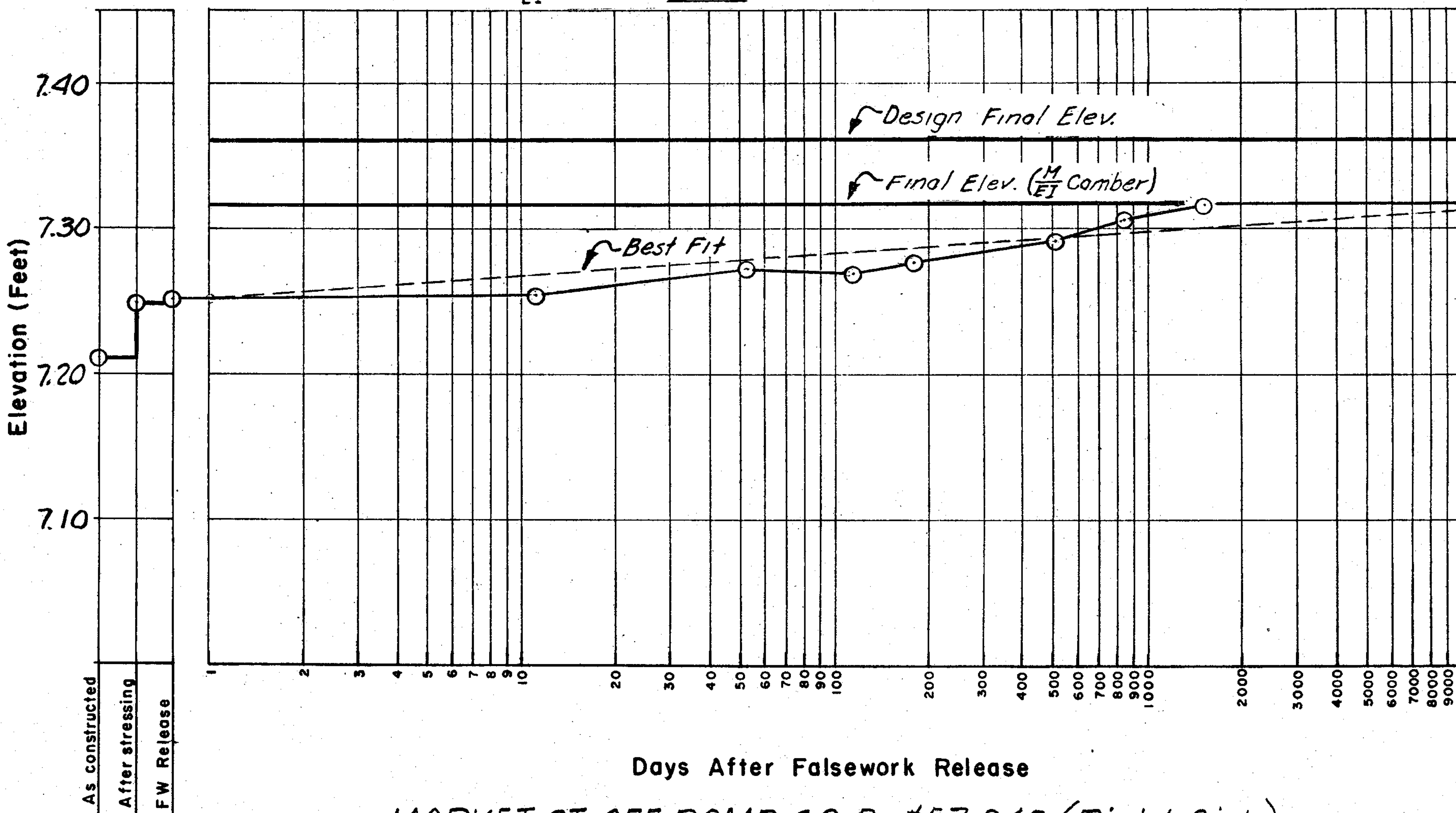
MARKET ST OFF-RAMP OC Br #57-842 (Right Side)

Figure 79

DEFLECTION AT ℄ OF SPAN

Span 3 Plan Camber = -0.15' Initial Elev. = 147.21 Span Length = 118.3'

Adj. $\frac{M}{EI}$ Camber = -.104



MARKET ST OFF-RAMP OC Br #57-842 (Right Side)

Figure 80