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Dynamic tests of a 55ft. Amdek bridge member, March 1956

Carl E. Ekberg Jr

Report on

DYNAMIC TESTS OF A 55 FT. MEMBER

for

Concrete Products of America Division
American-Marietta Company
Pottstown, Pennsylvania
Mr. S. L. Selvaggio, Chief Engineer

by

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Test Conducted by

Fritz Engineering Laboratory

Lehigh University

Bethlehem, Pennsylvania

March, 1956

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INTRODUCTION

This report contains the significant results of tests on a 55 foot pretensioned prestressed concrete bridge member under dynamic and static loads. The data is presented in this report in the form of graphs and tables, and certain pertinent conclusions with regard to the 55 foot member are indicated. At the end of the report final observations and conclusions are discussed relevent to a comparison of the 55 foot beam under dynamic loads and a 70 foot beam under static loads.

OBJECTIVES

1. To determine the feasibility of a rectangular box section for long span bridge members with the special intent of finding:

   (a) Behavior of the beam under working loads

   (b) Behavior of the beam under higher than working loads

with regard to deflection, presence of cracks, slip of strands, and ultimate load.

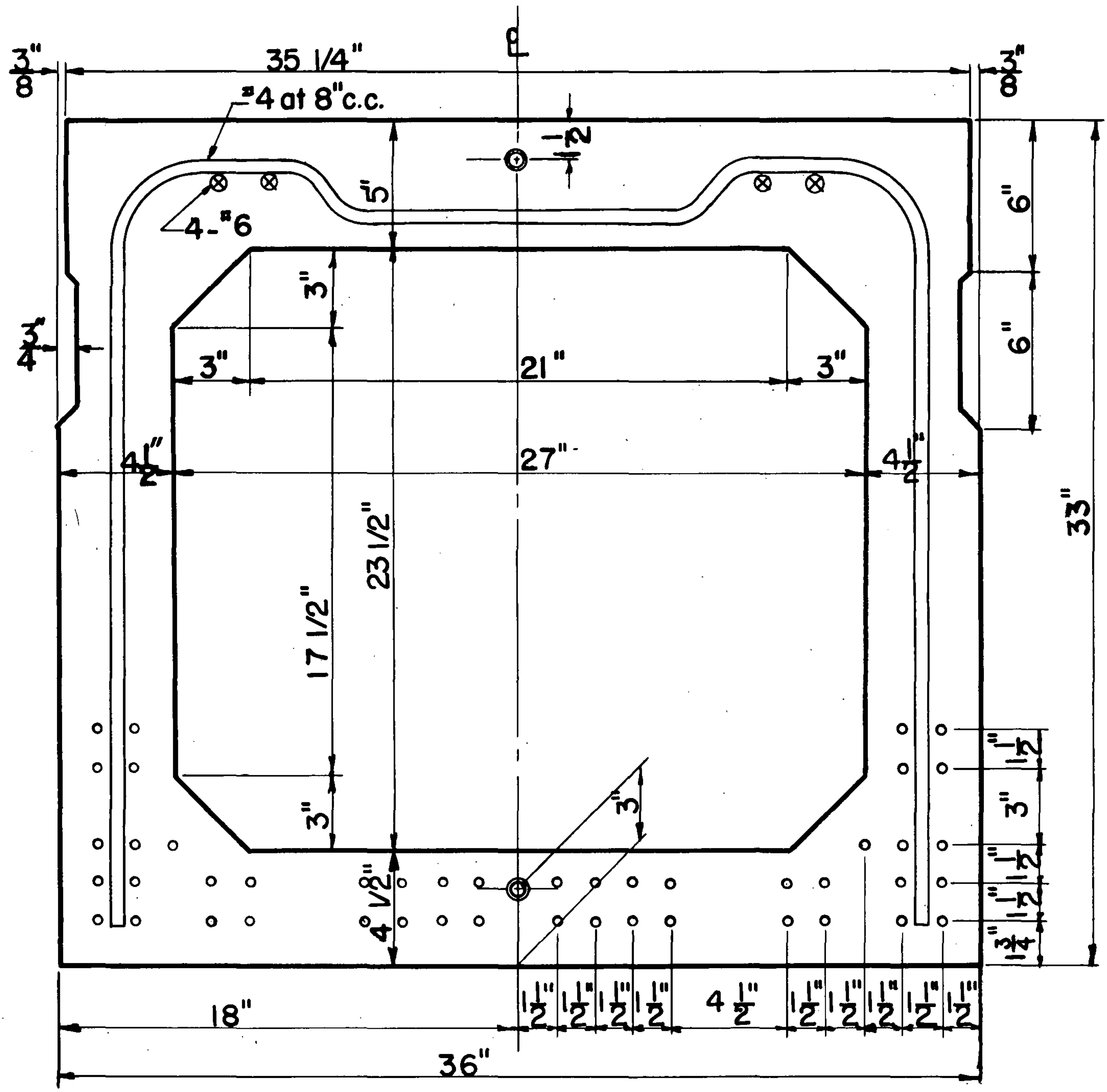
2. Compare the dynamic behavior of the 55 foot beam to the behavior under static load of a 70 foot beam previously tested.

DESCRIPTION OF THE BEAM

The beam was of pretensioned bonded design having overall dimensions of 55 ft. in length, 36 in. in width and 33 in. in depth. Rectangular hollows passed the full length except for two ft. solid portions at the ends and at midspan. The prestressing tendons consisted of 46 strands of 3/8 in. diam, and stressed initially to 150,000 psi. The conventional reinforcing steel consisted of inverted U-shaped No. 4 bars and four longitudinal No. 6 bars. The No. 4 bars were spaced at 8 in. centers and acted in a dual role as stirrups and as transverse flexure reinforcing for the top side of the section. The longitudinal bars likewise functioned in a dual role, as they passed the full length of the beam near the top fiber and served to minimize the opening of shrinkage cracks as well as to tie the system of U-shaped rods into one easily handled unit. A typical cross-section is shown in Figure 1.

Design

The design of the beam is based on the specifications of the Pennsylvania Department of Highways. An analysis of the beam is given in Appendix A (Part I) wherein the stresses at the equivalent static design load are calculated (See p.42).



SECTION A-A

Fig. I - Test Beam

It should be mentioned that the beam was originally designed as a span 70 ft. center-to-center of bearings. The analyses presented in Appendix A are patterned after the design calculations submitted by the manufacturer.

Materials

Concrete

The concrete was mixed in a one cu. yd. capacity mixer and was poured in a 25 deg F atmosphere. A high frequency internal vibrator was used to compact the concrete in the forms before application of a 30-minute vacuum treatment. The vacuuming was followed by five days of steam curing at a temperature of approximately 125 deg F. The mix had the following proportions on a cubic yard basis:

Cement - Type I (a) - 9 sacks
Wet Sand - (5% surface moisture) - 1239 pounds
Crushed Rock - (1% surface moisture) - 1876 pounds
Water - 27 gallons
Water:Cement - 4.06 gallons per sack.

Steel

The steel strand had a nominal diameter of 3/8 in. with an ultimate tensile strength of 250,000 psi. Each strand was tensioned individually by a calibrated hydraulic jack to a stress of 150,000 psi.

Manufacture

The beam was manufactured at the Pottstown, Pennsylvania plant of the Concrete Products Company of America on a prestressing bed 125 ft. in length. A second beam of 70 ft. length was poured on the same bed with a cross-section identical to that of the 55 ft. beam. The entire pouring operation was continuous and required a total of about three and one-half hours for both beams.

The schedule of manufacture is shown in Table 1.

Table 1 - Manufacturing Schedule for the Beams

Event	Date
46 strands tensioned to 150,000 psi	Jan. 24, 1956
Concrete placed	Jan. 25 (2 hr.)
Vaccuum process	Jan. 25 (30 min.)
Steam curing at 125 deg F	Jan. 25 - Jan. 30
Release of prestress	Jan. 30
Removal from bed	Jan. 31
Storage in plant at outside temp	Jan. 31 - Mar. 9
Removal to Fritz Engineering Laboratory for testing	Mar. 9

TEST PROGRAM

Pottstown Plant

The testing at the plant was conducted so as not to interfere with normal plant operation. The test work included the measurement of total beam shortening, slip of strands at release of prestress, and strains on the concrete.

Fritz Engineering Laboratory

The test work in the Laboratory involved the dynamic loading of the beam near the mid-span with measurement of corresponding vertical deflections, strains on the concrete as determined by both Whittemore and SR-4 gages, and crack patterns. The SR-4 and Whittemore strain measurements will not be treated in this report.

TESTING PROCEDURE AND TEST RESULTS

Pottstown Plant

Total Shortening

The total shortening of the beam upon release of prestress was measured near the top and bottom sides. This was achieved by inserting longitudinally two 1/2 in. steel pipes into the concrete with one located 1-1/2 in. below the top surface of the beam at its center and the other located 3 in. above the bottom surface of the beam at its center (see Fig. 1). Into each pipe a 5/16 in. round greased rod, 70 ft. long, was placed. Ames Dial indicators were independently mounted to the concrete on each end of the rods. The total change in length of fibers of the beam at these levels was thus indicated by a change in the readings of the dials.

Figure 2 shows the 5/16 in. rods projecting from one end of the beam near the top and bottom. It will be noted that the Ames Dial at the top is mounted on a steel rod which

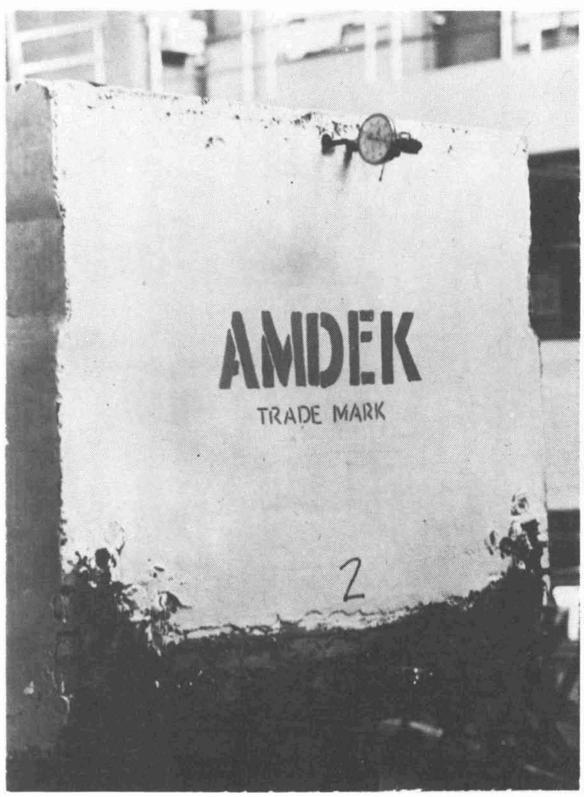


Fig. 2 - End View of Beam Showing Dials for Total Shortening

has been set firmly into a drilled hole in the concrete at the same level as the pipe itself which projects slightly from the concrete. The 5/16 in. rods were lubricated to such a degree that they could easily be pushed back and forth with the fingertips.

Figure 3 shows the total shortening after release of prestress over a period of 20 hours. It is surprising to note that the fiber $\overline{PQ}$ near the top of the beam shortened about half as much as the fiber $\overline{RS}$ near the bottom. Actually, if the beam had performed at this stage as a perfectly elastic and homogeneous body the fiber $\overline{PQ}$ should have shortened to a value of only about 0.01 in. The large amount of shortening along $\overline{PQ}$ suggests the possibility that a number of minute cracks were present in the top fiber, and these were closed upon release of prestress.

Loss of prestress may be approximated from the total shortening along $\overline{RS}$ (See Fig. 3) as follows:

$$\sigma = \left(\Delta_s + \frac{wy}{2E_cI}\int_0^L \left(Lx - \frac{x^2}{2}\right) dx\right)\frac{E_s}{L} = \left(\Delta_s + \frac{wyL^3}{6E_cI}\right)\frac{E_s}{L}$$

Letting $w = 580$ lb./ft.

$E_c = 5 \times 10^6$ psi

$I = 78{,}900$ in^4

$L = 55$ ft.

$y = 13.53$ in.

$E_s = 30 \times 10^6$ psi

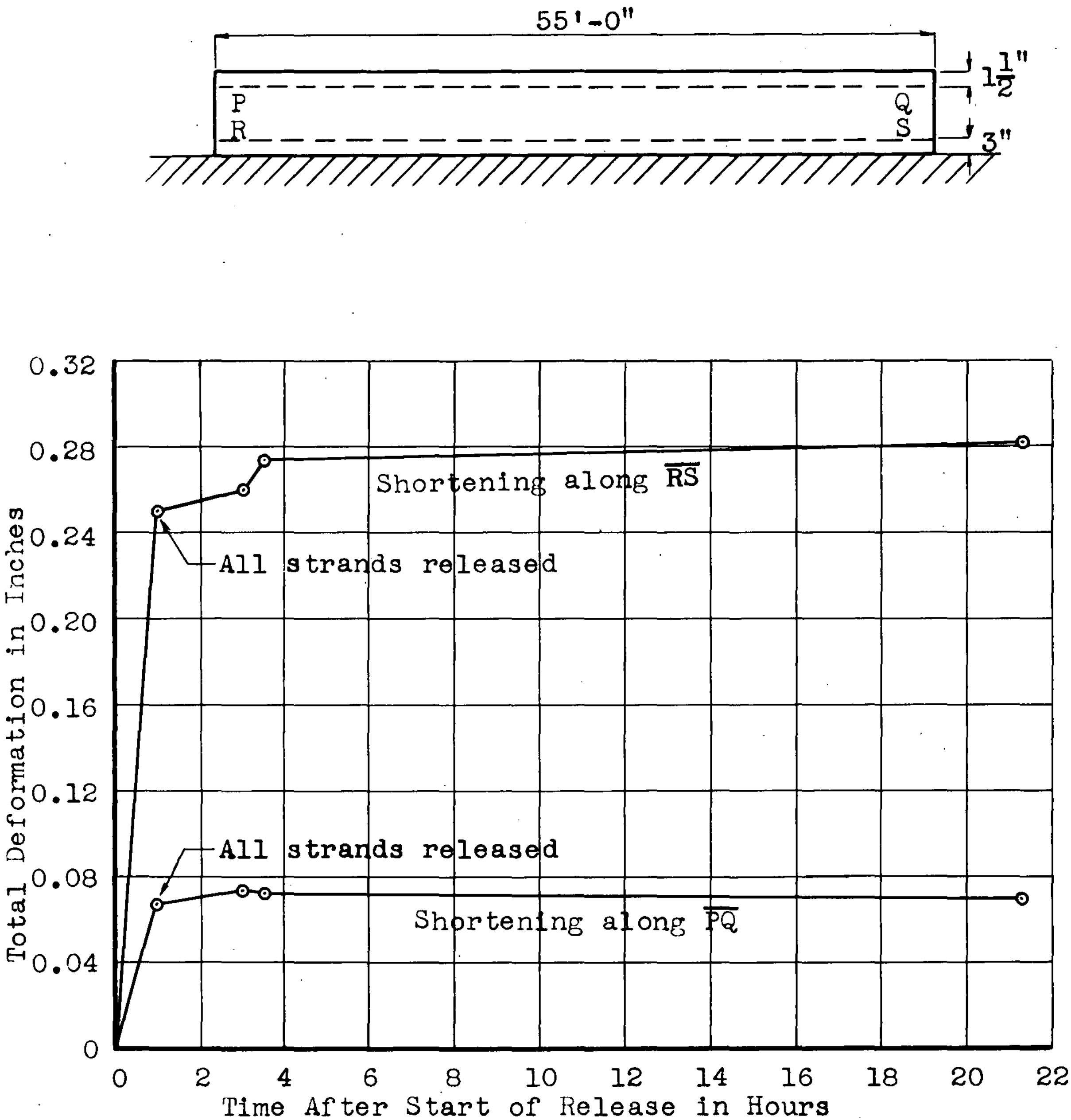


Fig. 3 - Total Beam Shortening Near Top and Bottom Fibers

We get for the second term a value of .08 in. and a measured value of Δ_s = 0.28 in. Substituting, a loss of prestress of

$$\sigma = 16{,}350 \text{ psi}$$

was obtained. This represents a loss of approximately 10.9 percent at one day after release of prestress.

Slip of Strands at Release

The movement into the concrete of the strands upon release of prestress was measured at one end of the beam only. In this case, three strands were selected because of space limitations; however, it is believed that typical values were obtained. The slip was measured by means of Ames Dials mounted on the strand with stems bearing against the concrete. Thus, any strand movement into the concrete was directly measured upon release of prestress.

The slip values for each of the four strands measured are 0.074 in., 0.054 in., and .054 in.: or an average of 0.061 in.. These results are about the same as have been obtained in the Laboratory in numerous tests which have been conducted during the past two years.

Fritz Engineering Laboratory

Test Setup

In Figure 4, the simply supported 55 foot beam is shown on the dynamic test bed, with the steel framework to which is attached the Amsler hydraulic jacks. These jacks applied the loads to the beam through transverse distribution beams, at points 4 ft. on each side of the midpoint. (The outside set of jacks are of larger capacity and were not in operation when this photograph was taken.) In the right foreground are the Amsler pulsators which pump the oil to the hydraulic jacks and control the oil pressure for either static or dynamic loading. The dynamic loads at each jack were applied at the rate of 250 cycles per minute.

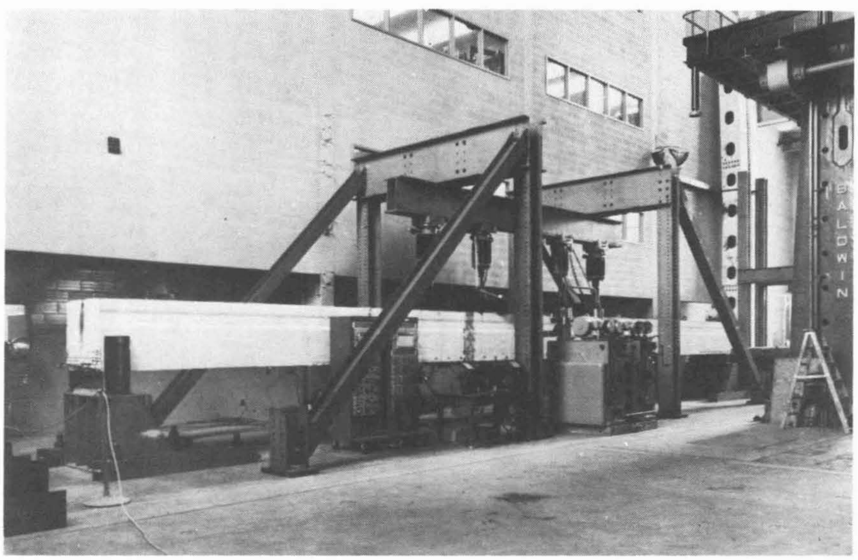
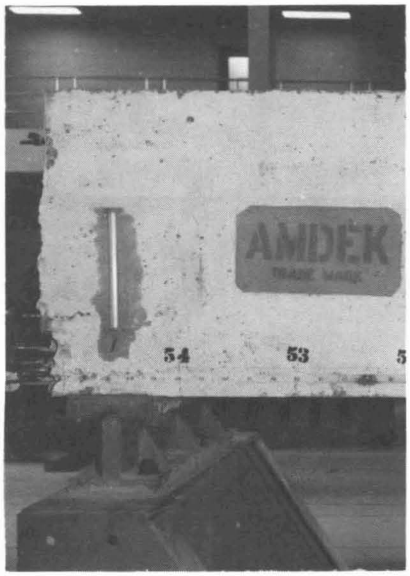


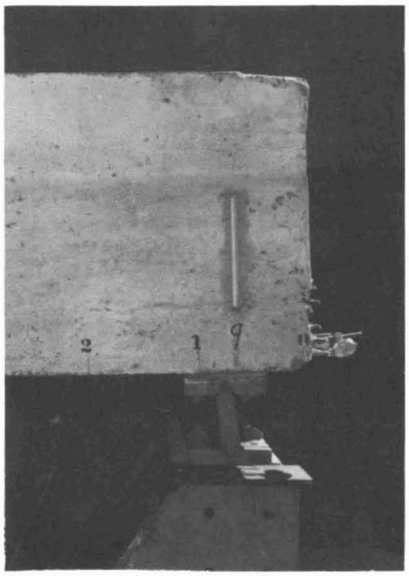
Fig. 4 - General View of Test Set-up

On a table to the left of the pulsators are the electrical resistance strain gage indicators used for measuring static loading strains. To the left of the table are the two Brush Dynamic Strain Recorders.

Figure 5 shows the details of the supports that were used for the beam. The right-hand view shows the heavy steel rocker at the expansion end of the beam. A layer of mortar was placed between the bearing plate and the bottom of the beam at each end.



Fixed End



Expansion End

Fig. 5 - Details of Supports

Static and Dynamic Loadings

In order to correlate the test of the 55 ft. beam with the 70 ft. beam tested earlier, it was necessary to calculate an equivalent static design load and a design dynamic load as shown below. In addition, other loadings were calculated and used in accordance with the sequence shown in Fig. 6.

Equivalent Static Design Load

This loading was determined by equating the dead load plus the jack load moment in the 70 ft. beam at a point 27 ft. from the left end to the dead plus jack load at the midspan of the 55 ft. beam. This calculation, shown diagramatically in Fig. 7, indicates that two jack loads of 20,500 lb. each will give a static moment which is approximately equivalent to the design moment of the 70 ft. beam. Fig. 6 shows that Static Tests 1 - 4 inclusive were conducted with equivalent static design loading.

Design Dynamic Load

This is the varying load applied by the Amsler jacks at two load points as shown in Fig. 15. The values of 1700 lb. minimum and 5500 lb. maximum were calculated by an approximate method, and were to be of such magnitude that when applied at a frequency of 250 cycles per minute would cause the same

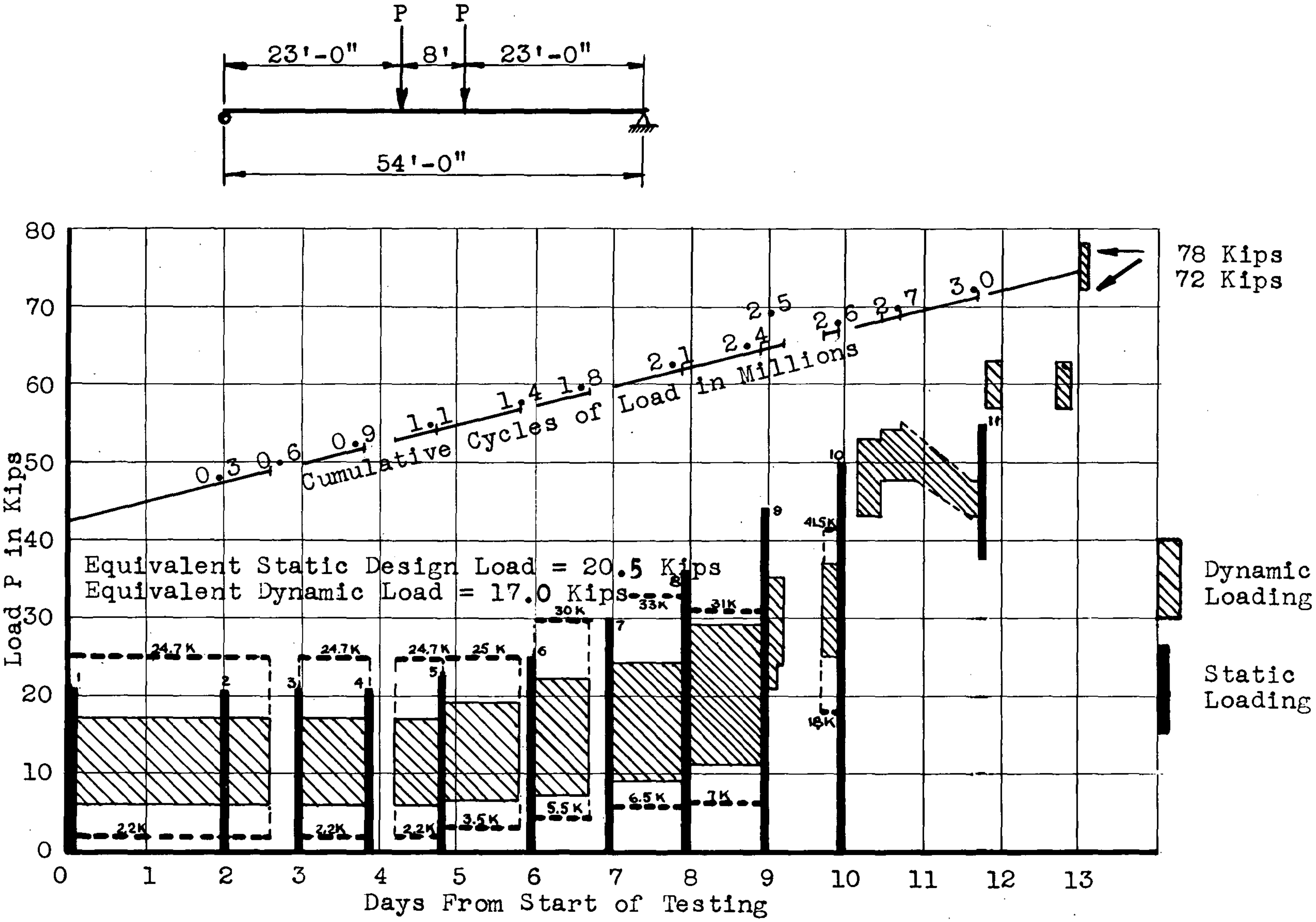
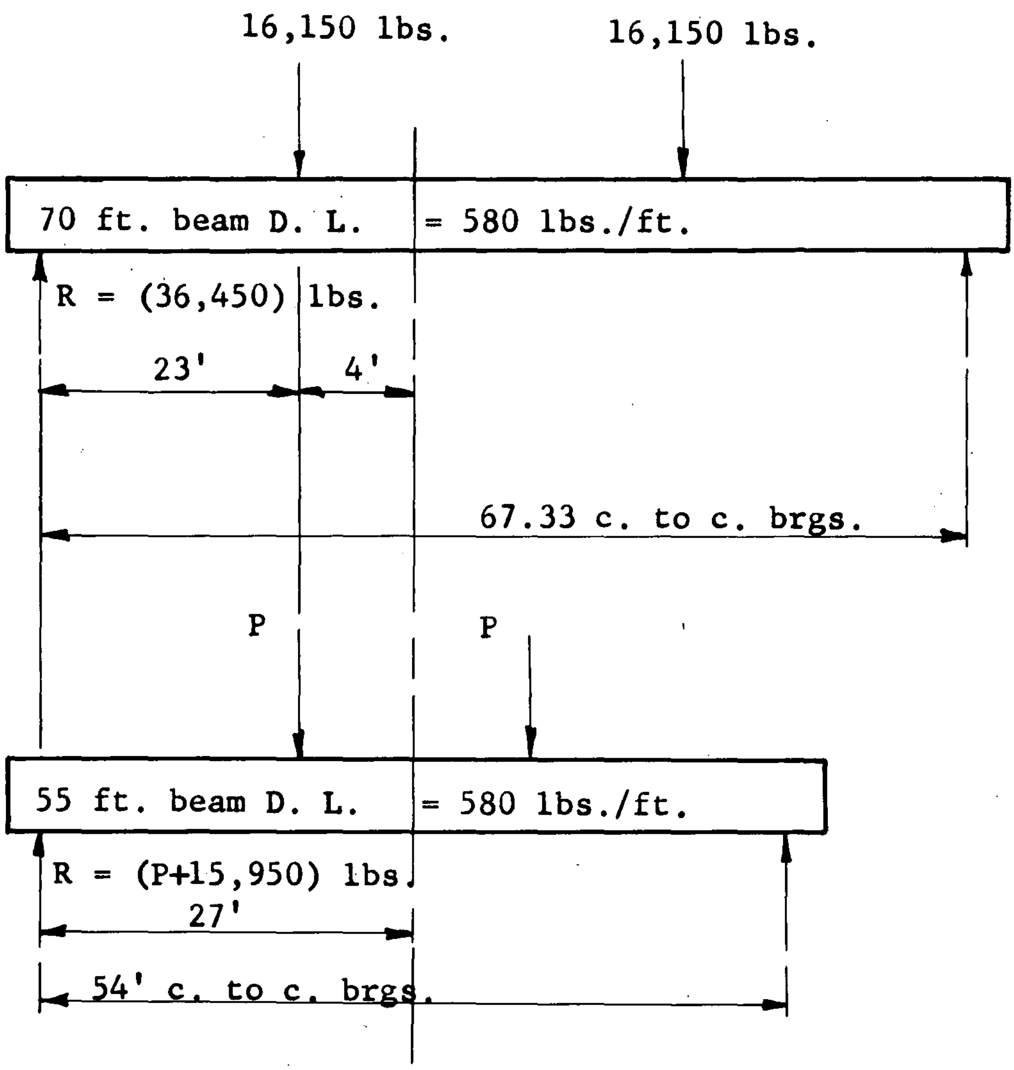


Fig. 6 - Loading Sequence for 55 Ft. Beam



Moment due to dead load and jack loads at center-line of 55 ft. span = Moment due to dead load and jack loads at 27' from left support of 70 ft. span

Using the left free bodies we have:

$$(P+15{,}950)27-4P-580(27.5)\left(\frac{27.5}{2}\right) = 36{,}450(27)-16{,}150(4)-580(28.33)\left(\frac{28.33}{2}\right)$$

Solving, P = 20,500 lbs. (Equivalent Static Design Load for the 55 ft. Beam)

Fig. 7 - Diagram and Calculations for Equivalent Static Design Load

maximum deflection in the beam as that due to the equivalent static design load. Actually, the maximum dynamic deflection produced during the test was approximately 0.1 in. greater than the maximum deflection produced by the equivalent static design load, as will be noted in Fig. 8. The design dynamic load was applied during the first four days of testing with some interruptions due to mechanical difficulties. The design dynamic load is shown by the shaded areas in Fig. 6, and is seen to extend for 1.1 million cycles. The heavy dashed lines in Fig. 6 shows the limits of the effective loads, which if applied gradually at the jack points, would produce deflections equal to the measured dynamic deflections. The effective loads exceed the jacking load by an amount approximately equal to the effect of the inertia forces.

Other Loadings

Fig. 6 shows that after 1.1 million cycles of design dynamic load has been applied, the loadings were increased in stages at each 300,000 - 400,000 cycles. The equivalent static loads were successively increased to 22,500 lbs., 25,500 lbs., 30,000 lbs., 36,000 lbs., 42,000 lbs., 50,000 lbs., and 55,000 lbs., (in Static Tests 5-10 inclusive) and dynamic loads were calculated by the method previously used.

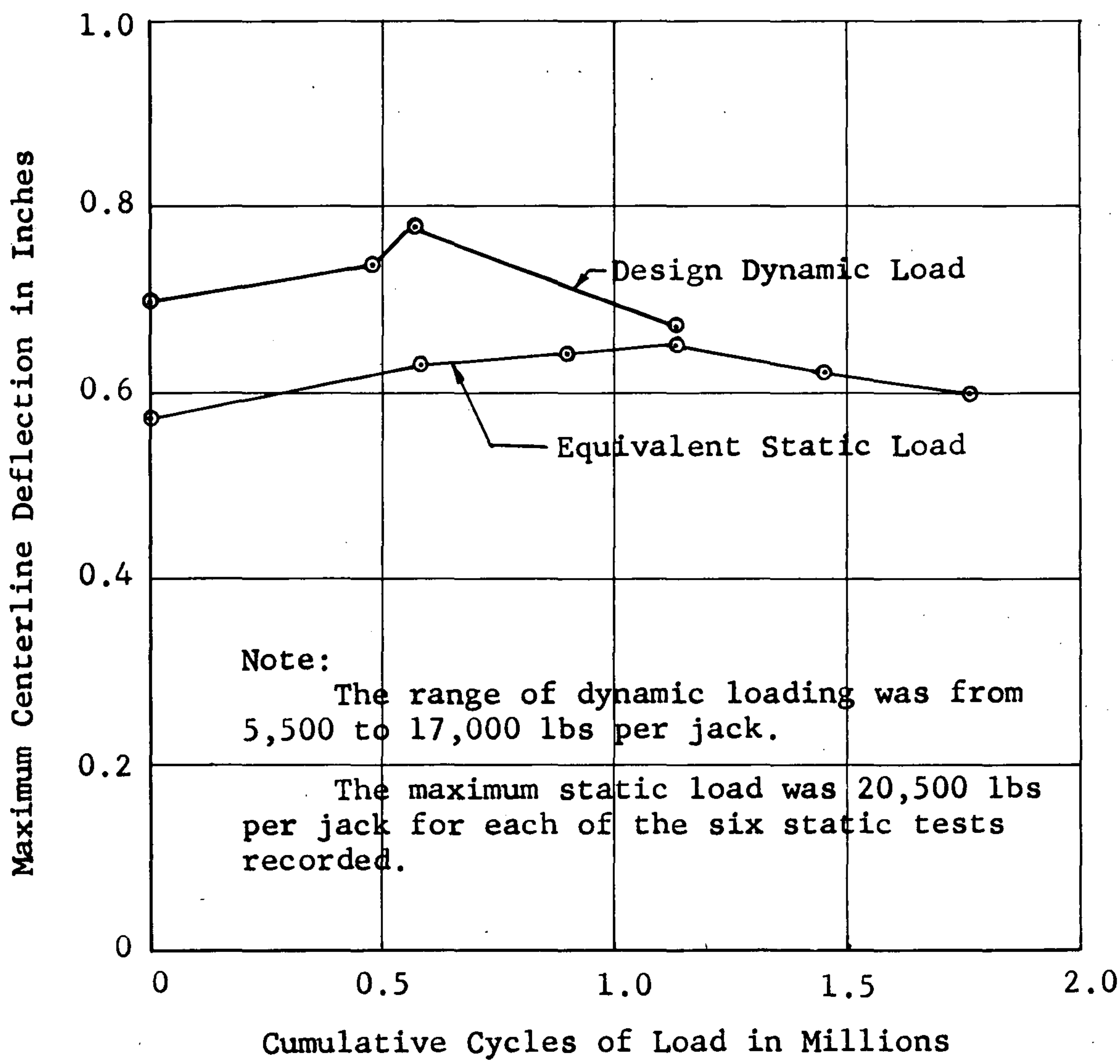


Fig. 8 - Maximum Deflections Under Equivalent Static Load and Design Dynamic Load

The effective loads were calculated as before, using the data from the corresponding static tests in conjunction with the measured deflections under the dynamic loading. The dynamic loads, corresponding to the equivalent static loads given above, were applied by the Amsler jacks at the rate of 250 cycles per minute in the following ranges: 6,000 - 19,000 lbs., 7,000 - 22,000 lbs., 9,000 - 24,500 lbs., 11,000 - 29,000 lbs., and 21,500 - 35,500 lbs..

Deflections

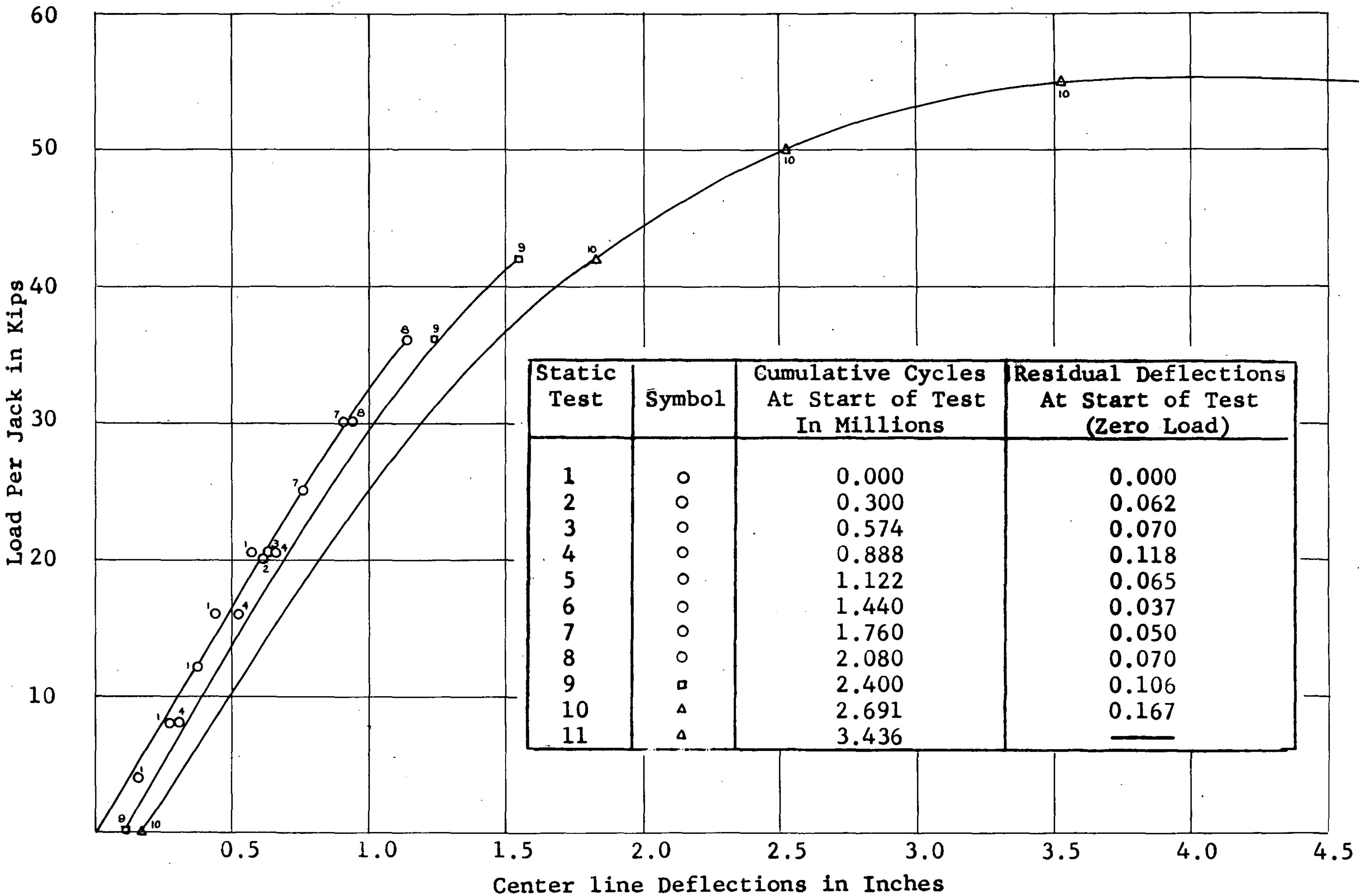
Figure 6 shows that the testing program consisted of periods of cyclic loading followed by static tests until failure of the beam occurred. Static tests numbered 1 through 11 were run after 0, 0.3, 0.6, 1.1, 1.4, 1.8, 2.1, 2.4, 2.6, and 3.0 million cumulative cycles of repetitive loading. The results of all tests will be subdivided into two parts for discussion, namely static tests and dynamic tests.

Static Tests - The purpose of the static tests was to determine if any change in the elastic properties of the beam had occurred due to the repetitive loading. Figures 8 and 9 show very conclusively that very little change, if any, occurred during the first million cycles of loading.

Figure 9, shows all pertinent load-deflection values based on the data from Static Tests 1-11 inclusive. Shown in tabular form are the cumulative cycles and residual deflections at the start of each test. The figure shows that the stiffness of the beam remained practically constant throughout the first 8 static tests. In Statics Tests 9, 10, and 11 for higher loadings, there was the expected deviation from the linear relationships. The influence of cracks on the beam stiffness is quite pronounced at the higher range of loading, however it seems generally true that the number of load repetitions had little or no effect on the beam stiffness.

<u>Dynamic Tests</u> - The dynamic deflections were measured by means of a vernier caliper mounted on a stand so that the upper and lower jaws could come in contract with a steel dowel on the beam (See Fig. 10). The dowel was set in a drilled hole near the lower fiber of the beam and was perpendicular to the vertical side of the beam. The vernier was adjusted so that the dowel moved up and down within the two jaws, and a reading was taken when the moving dowel was in slight contact.

Figure 11 shows principally the measured deflections under dynamic loading at midspan. It can be seen here that within each range of loading the difference between maximum and



Static Test	Symbol	Cumulative Cycles At Start of Test In Millions	Residual Deflections At Start of Test (Zero Load)
1	○	0.000	0.000
2	○	0.300	0.062
3	○	0.574	0.070
4	○	0.888	0.118
5	○	1.122	0.065
6	○	1.440	0.037
7	○	1.760	0.050
8	○	2.080	0.070
9	□	2.400	0.106
10	△	2.691	0.167
11	△	3.436	—

Fig. 9 - Static Deflections at Various Stages of the Test Program

minimum deflections is fairly constant for every load condition. There is, for the most part, a linear relationship between variations of load and corresponding variations of deflection. Apparently, within design load and up to 24 kips maximum jack load, the beam behaves elastically in a manner which is independent of the number of cycles.

Fig. 10 - Measurement of Deflections Under Repetitive Loading

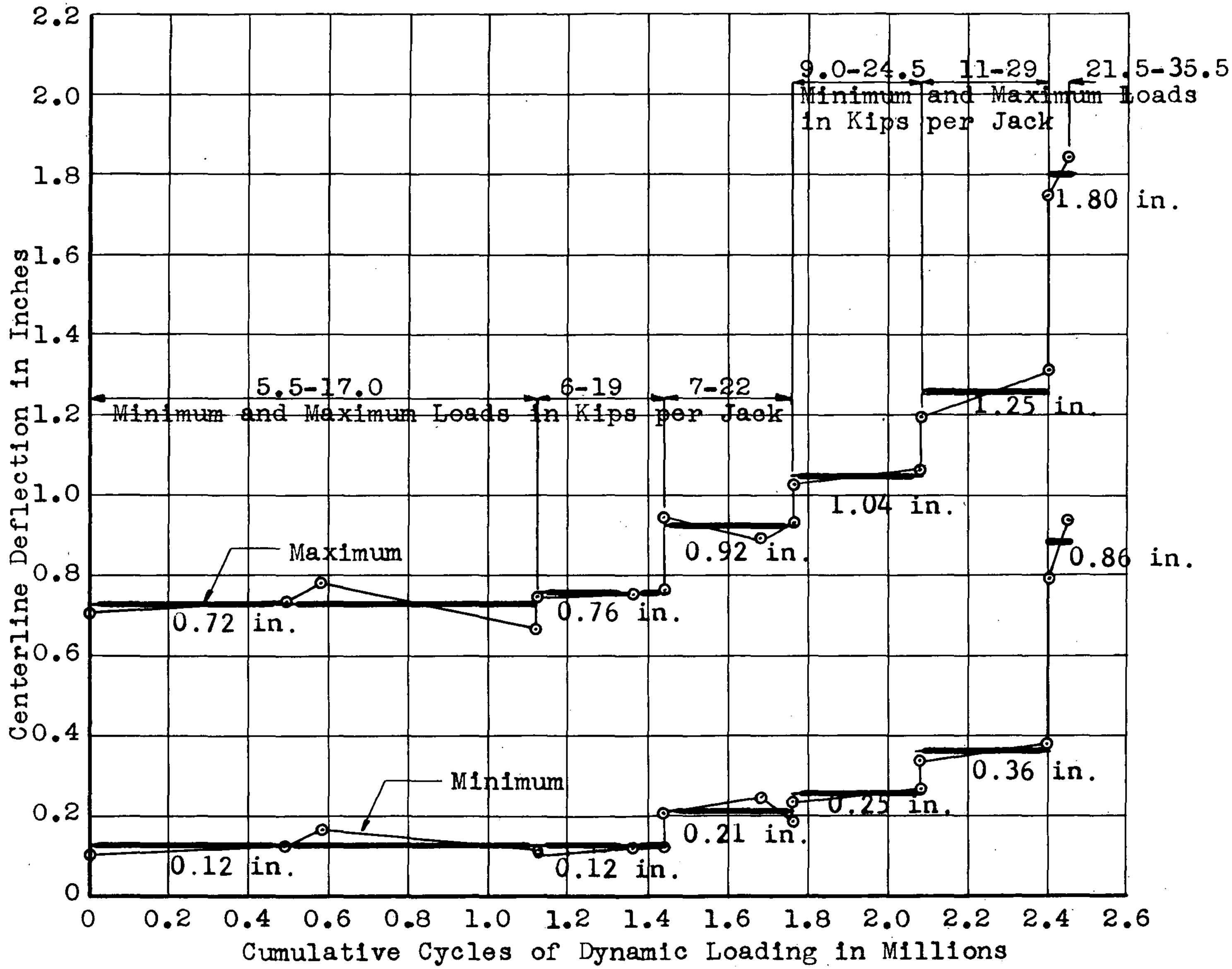


Fig. 11 - Dynamic Deflections at Various Stages of the Test Program

Strand Slip

The method of measuring end slip of strands is shown in Fig. 12 below. Four representative strands were selected at each end of the beam and one Ames Dial was mounted at each strand. The dials were clamped to rods set in drilled holes in the concrete and each dial stem was brought to bear against a separate strand. All four strands were in the lower row of strands where the greatest chance of slippage would occur. The ends of the four strands were first burned with a torch to fuse the individual wires together, and then later ground smooth with an emery wheel to provide a good contact surface for the stem. The dials would measure displacements as small as 0.001 in..

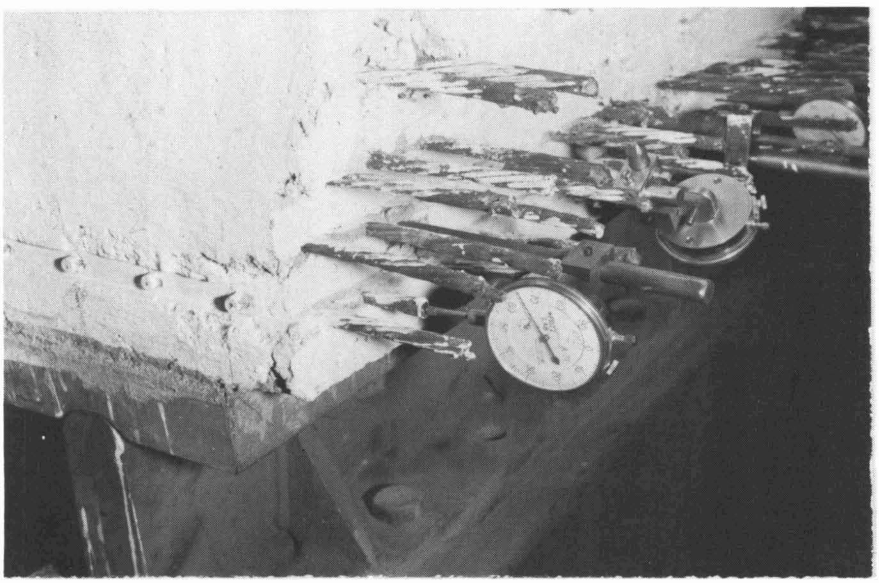


Fig. 12 - View Showing Method to Detect Strand Slip

Strand slip was not detected at any time during the various stages of the loading. The Ames Dials were kept in place during all of the repetitive loading operations, and during the final stages of loading to ultimate. Fortunately, at ultimate load, the beam remained on the supports and none of the dials were jarred loose.

CRACKING CHARACTERISTICS COMPARED WITH 70 FT. BEAM

In this section, the cracking characteristics of the beam are compared with the cracking characteristics of the similar 70 ft. beam tested previously. It is seen in Fig. 6 that the shorter beam has a smaller maximum dead load moment (212 kip-ft.), and there is only 8 ft. between load points. The maximum dead load moment for the 70 ft. beam is 328 kip ft; the distance between load points is 21 ft.-4 in.. It will be recalled that the beams were cast on the same prestressing bed so that the tendons, consisting of 46 stress relieved strands of 3/8 in. dia (Fig. 1), could pass through both beams and give the same prestressing force.

Description of Loading

The loading diagram for the static test of the 70 ft. beam is shown in Fig. 13, and requires little explanation. The heavy black lines indicate the gradually applied loading and it is seen that there were four sustained loadings applied during the testing procedure as denoted by the cross-hatched areas. The equivalent design load was 16.15 kips per jack* and the load at initial cracking was 35.5 kips per jack.

The loading sequence for the dynamic test of the 55 ft. beam is shown in Fig. 6 and has been previously discussed. Crack measurements were made only during the static tests inbetween the periods of repetitive loading.

Method of Crack Measurement

All cracks were carefully marked and measured on one side of the beam only. A calibrated microscope made possible the accurate measurement of crack widths to the nearest 0.001 in. as seen in Fig. 14. The readings on cracks were taken at horizontal reference lines AB and A'B'; located 2-1/2 in. above the bottom fiber and 2-1/2 in. below top fiber, respectively, as shown in Fig. 15.

* See Appendix A - Part III

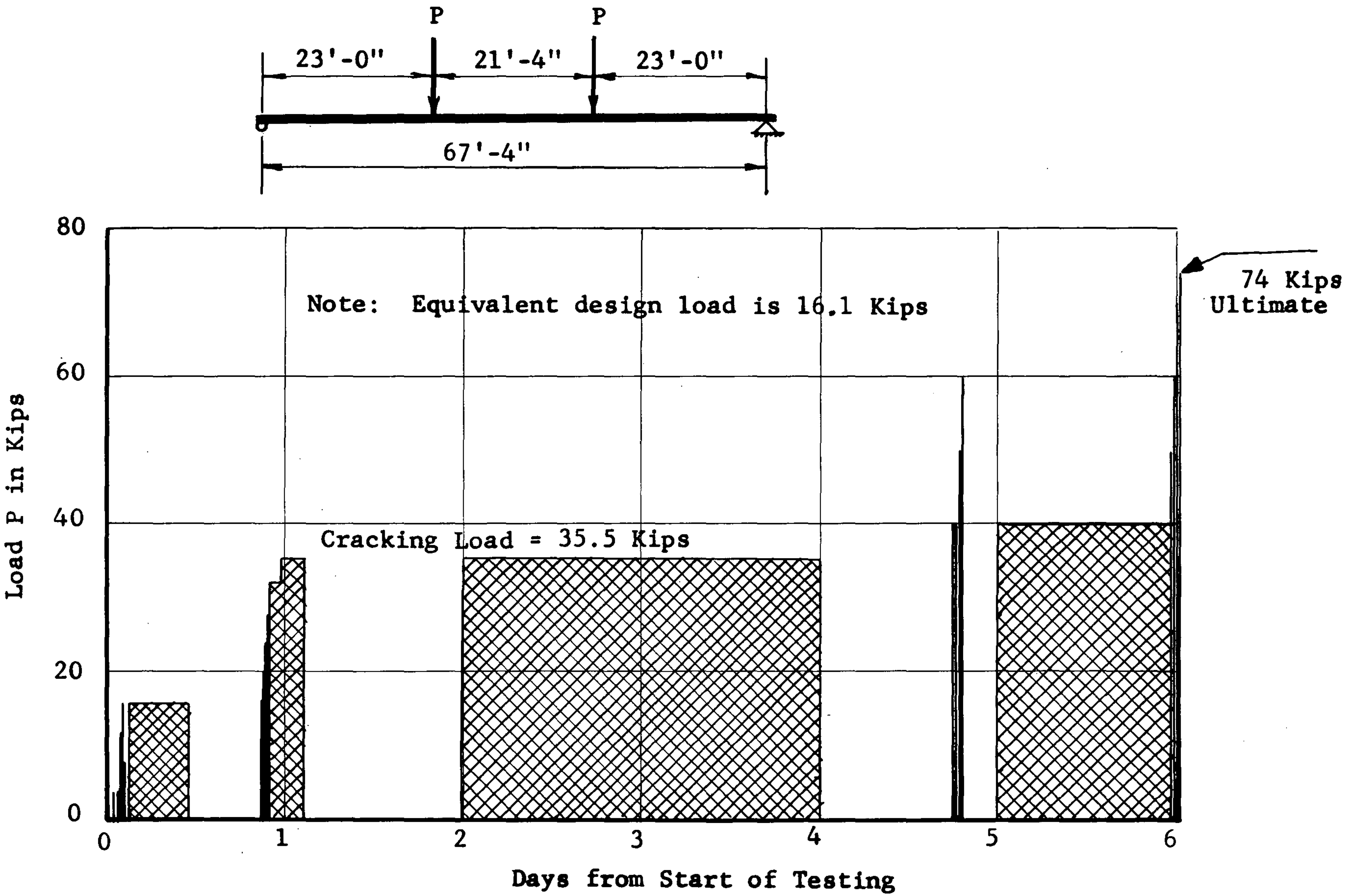


Fig. 13 - Loading Sequence for Static Test of 70 Ft. Beam

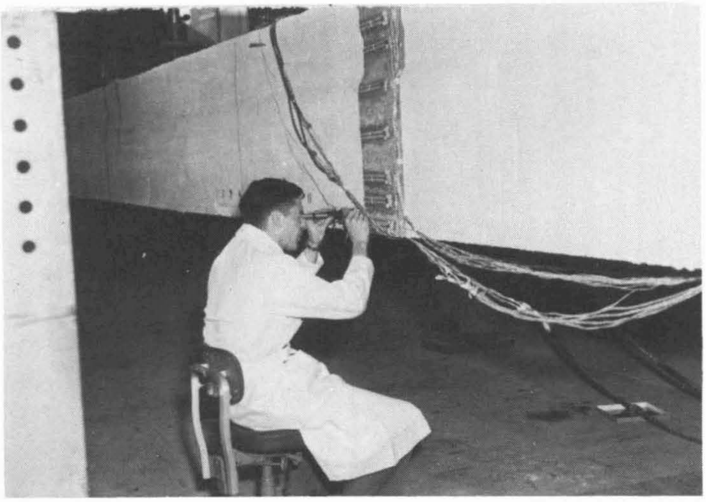
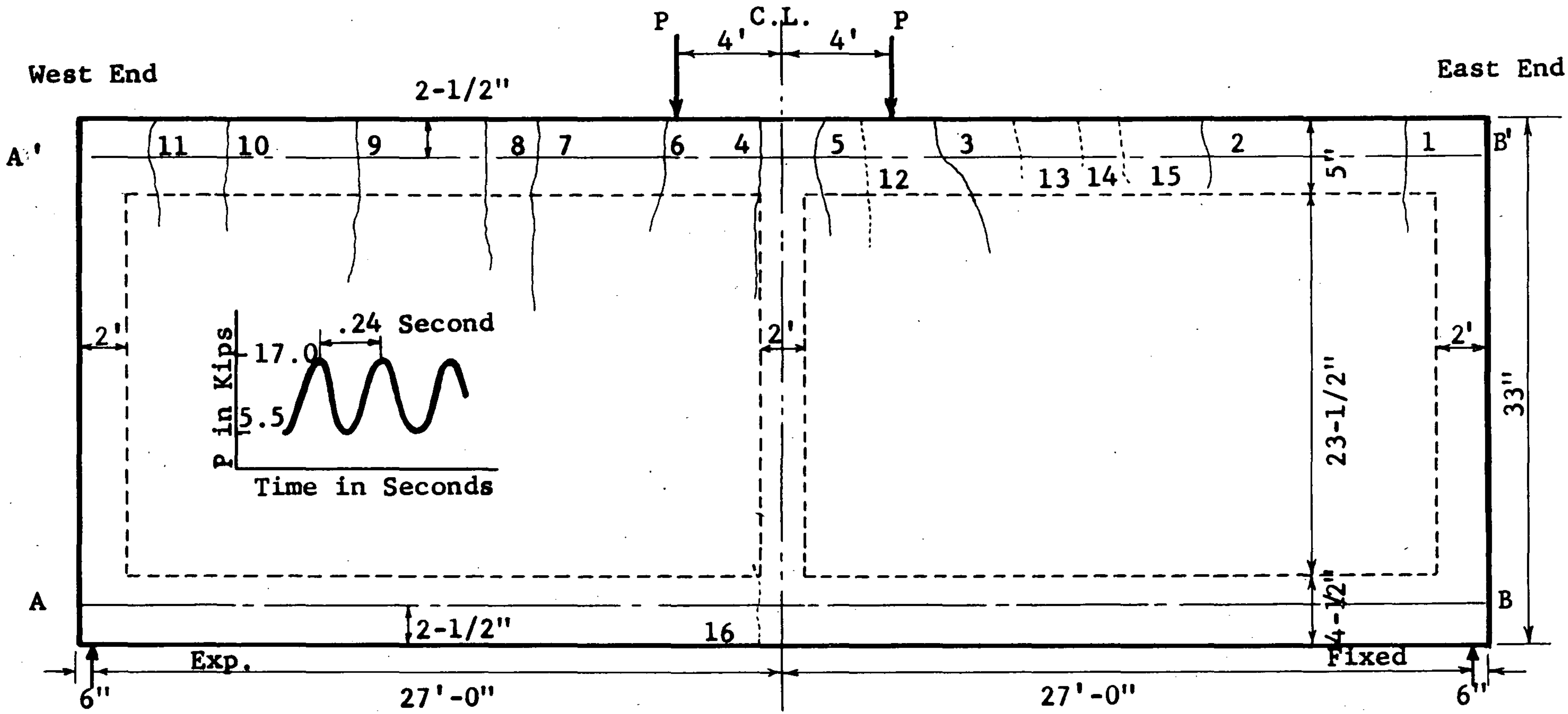


Fig. 14 - View Showing the Method of Crack Measurement.

In other words, the width of each crack at various static loads was recorded at the point where the crack crossed the horizontal reference line. In all cases, for both beams, the crack measurements were made only during the static tests at the maximum applied load of each run.

Appearance of First Cracks

For the 55 ft. beam, the first cracks were detected on the fourth static test after the application of 900,000 cycles of load as shown in Fig. 15. At this stage of loading a total of 16 small cracks were discovered of which 15 were located at various points along the top fiber. The top fiber cracks, as well as the one crack on the bottom fiber, were observed to pass over the entire width of the beam. Cracks numbered 1-15 inclusive were located along the top fiber, and crack 16 was discovered near midspan in the bottom fiber.



Vert. Scale: 1 in: = 10 in.
Horz. Scale: 1 in. = 6 ft.-3 in.

Note: Cracks 1-15 inclusive were closed upon application of equivalent static design load of 20.5 kips per jack.

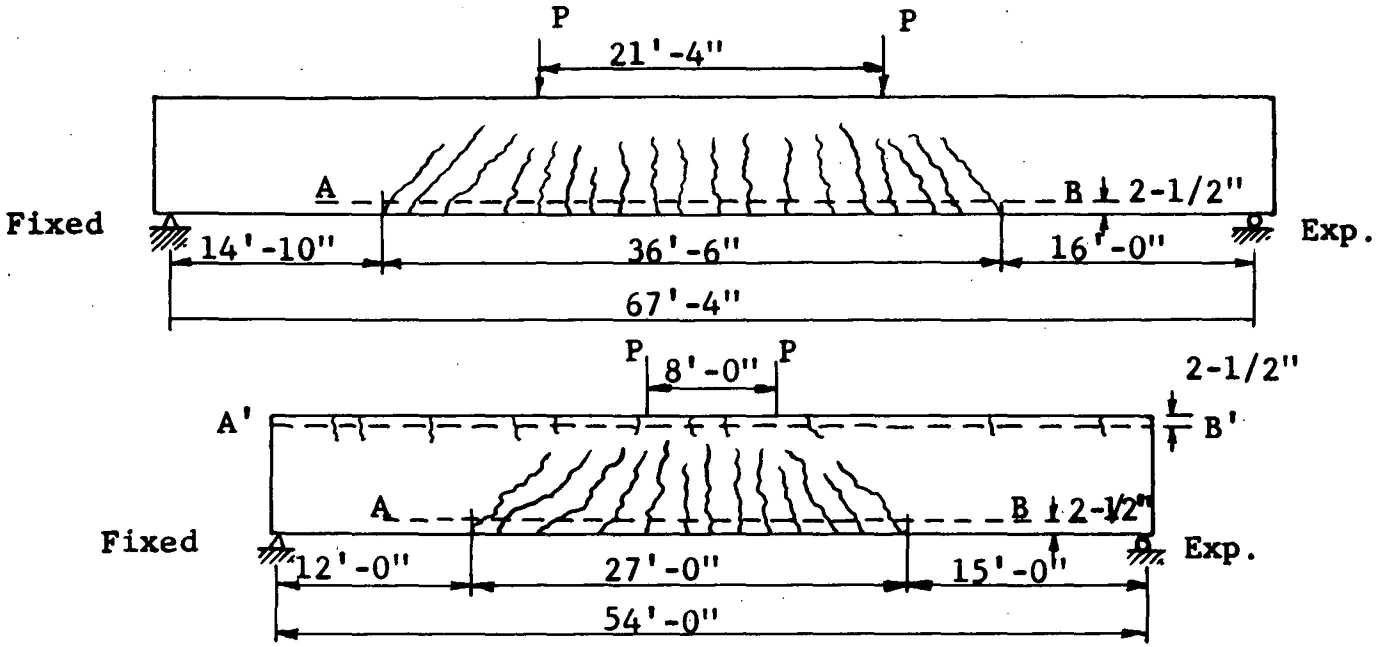
Fig. 15 - Crack Pattern after Application of 900,000 Cycles of Design Dynamic Load

Concerning the top cracks, it should be mentioned that they were only visible at the no-load stages of the test, and no additional cracks developed in this region with further testing. The average width of the top cracks was 0.004 in., and the average length was 9.1 in.; the maximum width was 0.012 in. and maximum length was 12 in. During all stages of testing, moreover, there was no apparent relationship between crack size and load intensity or repetitions of load. The first bottom crack to appear was located near midspan and measured approximately 0.001 in. in width under maximum load at fourth static test.

Discussion of Crack Measurements

Figure 16 shows the position and general appearance of the crack pattern of both the 55 ft. and 70 ft. beams exclusive of top fiber cracks. All cracks were nearly vertical in the region where crack widths were measured and became less in width at points higher up the side of the beam.

Figs. 17, 18, and 19 show the number of cracks, average crack width, and maximum crack width plotted as ordinates against total maximum applied moment as abscissa. The solid line represents the 70 ft. beam which was tested statically, and the broken line represents the 55 ft. beam. In the case of the 55 ft. beam, the crack measurements were made during Static Tests No. 6 through 10 inclusive.



Note: Crack Widths Measured Along Line AB

Fig. 16 - Position and General Appearance of Crack Patterns

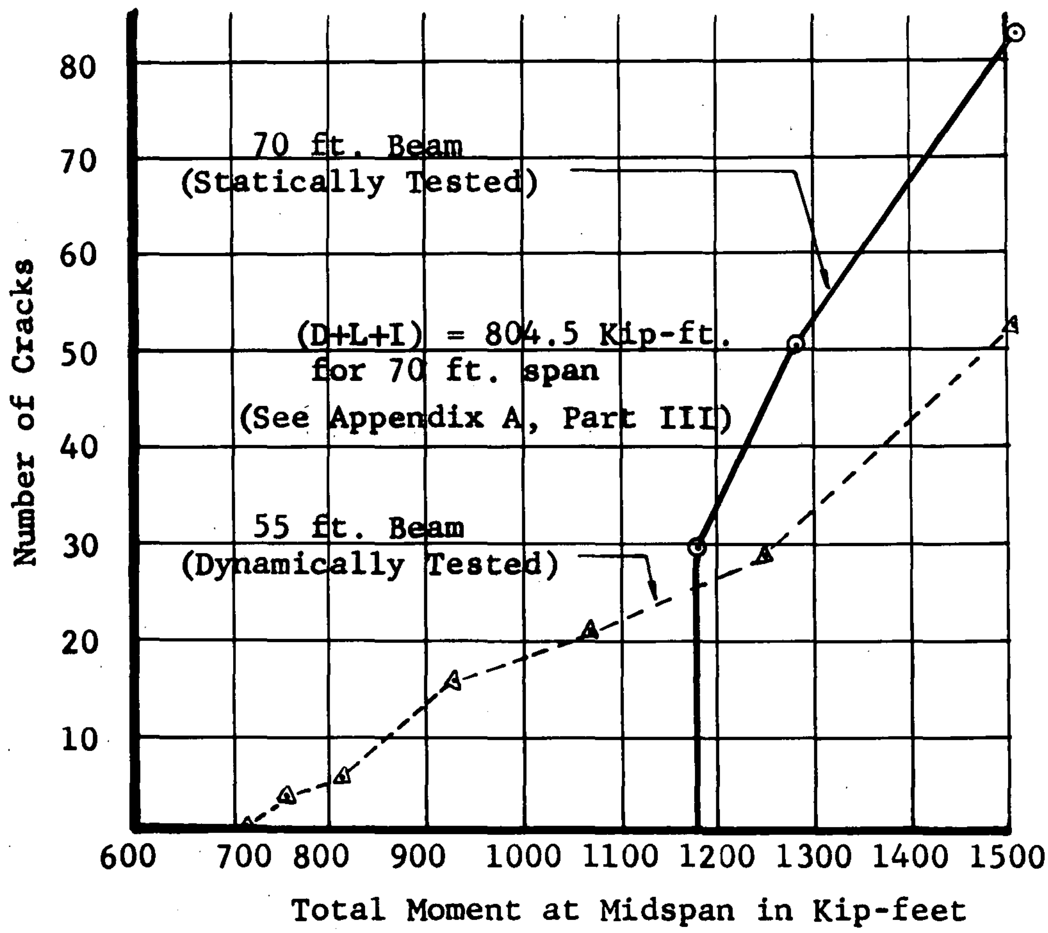


Fig. 17 - Number of Cracks vs. Total Moment

Fig. 17 shows a trend which is typical of all three graphs in that the first crack appeared at rather low load for the 55 ft. beam, but there was some tendency toward convergence of the two curves at the higher loading. The total number of flexural cracks in the 70 ft. beam was found to be 83 at the highest moment of 1501 kip-ft. at which cracks were measured. The total design moment (dead load plus live load plus impact) is 804.5 kip ft for the 70 ft beam. For the case of the 55 ft. beam the number of cracks at 1501 kip ft. moment was found to be 52, however, it seems reasonable to believe that the number of cracks might have been about the same had the two beams been of the same length.

Fig. 18 shows the relationship between the average crack width and the total applied moment for both beams. It is seen that the two curves converge quite closely at the cracking moment of the 70 ft. beam, however, at the largest moment they have diverged somewhat to values of 0.007 in. and 0.005 in. for the 55 ft. beam and the 70 ft. beam respectively.

Fig. 19 shows the relationship between the maximum crack width and the total moment. These curves reveal very strikingly the excellent behavior of the two beams, especially when one recalls that a crack may not be considered to be detrimental (as far as corrosion is concerned) until it reaches

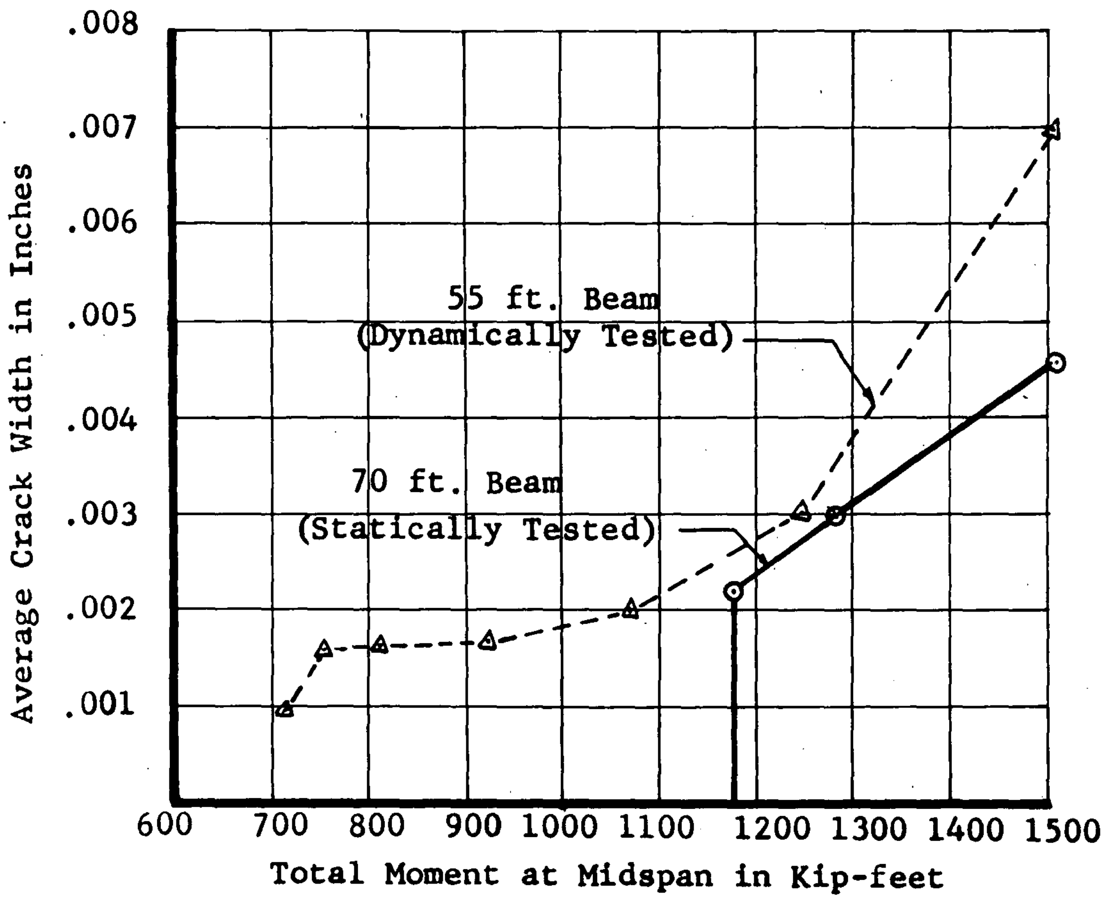


Fig. 18 - Average Crack Width vs. Total Moment

a width in excess of 0.010 in.. The critical width of 0.010 in. was reached at moments of 1340 and 1440 kip ft for the 55 ft. beam and 70 ft. beam, respectively. Here again we note a close correlation between the behavior of the two beams under high moment.

As a final bit of evidence of similarity of behavior of the two beams, it should be pointed out that the ultimate moments were practically the same. The total moments were 2060 kip-ft. and 2031 kip-ft. for the 70 ft. and 55 ft. beams, respectively.

Fig. 20 shows the crack pattern for the 55 ft. beam at a loading of 55,000 lb. at each load point. This indicates a strong similarity with the corresponding composite photograph of the 70 ft. beam (See Fig. 21 in Appendix A).

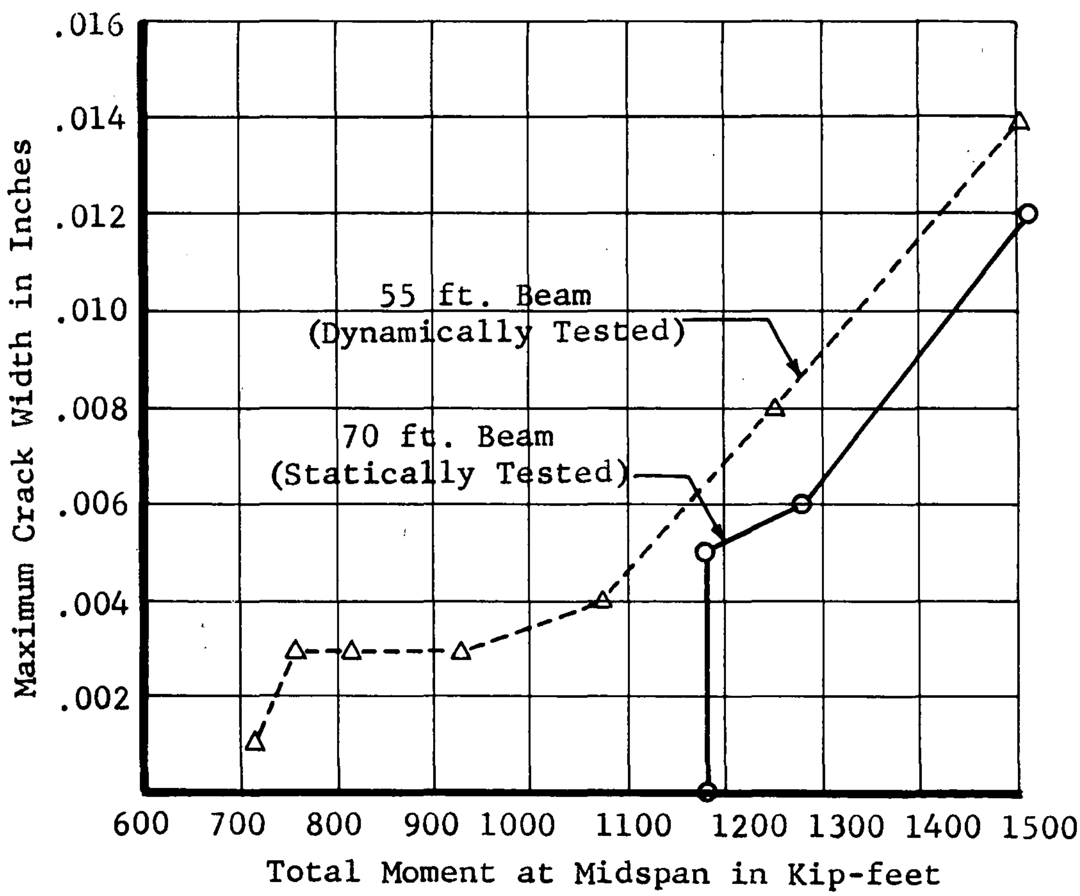


Fig. 19 - Maximum Crack Width vs. Total Moment

Fig. 20 - Crack Pattern with 55,000 lb. at Each Load Point After 2,900,000 Repetitions.

CONCLUSIONS

The test program proved the high resistance to fatigue of the prestressed pretensioned 55 ft. box-section member, manufactured under the trade name of "AMDEK". This can best be shown by comparisons with the static tests of a longer (70 ft.) member of the same type. The most important of these comparisons are listed below along with other observations concerning the general behavior of the 55 ft. beam.

1. The ultimate bending moment of the 55 ft. beam was not affected by the application of 3,000,000 load cycles.

Dynamic Test: (55 Ft. Beam)	Ultimate Moment	= 2060 kip ft
Static Test: (70 Ft. Beam)	Ultimate Moment	= 2030 kip ft

The above values are slightly in excess of

$$2.50\ (D+L+I)$$

where the value of (D+L+I) represents the total design moment for a span measuring 70 ft. center-to-center of supports (see Appendix A, Part III).

2. The mode of failure of both beams was essentially the same, that is they failed by crushing within the concrete compressive zone without any sign of bond disturbance. This is significant for the dynamic test, since one might expect, for example, a spalling of the concrete cover surrounding

the tendons. A careful inspection after ultimate loading revealed that the strands remained bonded in the case of both the 55 ft. beam and the 70 ft. beam.

It should be emphasized that the tests on the 55 ft. beam were dynamically severe only up to approximately one-half of the ultimate load since the deflection amplitude was limited by the testing machine. As the load-deflection diagram flattens out with higher loads, the load amplitude necessarily becomes small with the limited deflection-amplitude of the Amsler equipment.

3. The bending stiffness of the 55 ft. beam did not appreciably decrease during the first 1,100,000 cycles alternating from one tenth to the full design load. The bending stiffness maintained itself at about the same relative value as for the 70 ft. beam even though a few small hair cracks in the smaller beam were detected after 900,000 cycles. The constancy of the bending stiffness can be observed from the various incidental static tests in Fig. 9 of this report. The bending stiffness remained practically unchanged after 2,100,000 cycles and the application of

$$1.5\ (D+L+I).$$

4. The first microscopic crack in the tension zone could be detected after 900,000 cycles of design load. The

ratio of the dynamic to the static cracking moment as determined by the 55 ft. and 70 ft. beam tests, respectively, is thus

$$\frac{M_{cr\text{-}dyn}}{M_{cr\text{-}stat}} \overset{\text{Approx.}}{=} 0.6$$

This is in full agreement with certain theoretical considerations, involving the behavior under fatigue stresses of plain concrete, which may be applied to this particular case.

5. The premature appearance of cracks (as mentioned above), did not create any detrimental influence on the structural behavior of the beam. This can clearly be seen from the various comparisons between crack developments of the 55 ft. and 70 ft. beams. In the case of the 70 ft. beam, as many as 30 cracks formed almost instantaneously at the cracking load; whereas in the case of the 55 ft. beam (dynamically tested), the cracks were built up gradually in number and width. Figs. 17 and 18 show that although the cracking started earlier for the 55 ft. beam, it reached approximately the same level at the higher loads as in the case of the 70 ft. beam.

6. For the 55 ft. beam a maximum crack width of 0.01 in. was reached at a total moment of

1.62 (D+L+I).

The corresponding value for the 70 ft. beam was

1.75 (D+L+I).

A crack width of 0.010 in. is considered critical from the standpoint of corrosion of steel tendons.

7. No end-slip of the strand occurred at any stage of the test including ultimate load. This statement is true for both the 55 ft. beam and the 70 ft. beam.

8. The "AMDEK" members of 55 ft. and 70 ft. length, as tested in the Fritz Engineering Laboratory, are structurally sound with respect to both static and dynamic loading; and furthermore they exceed all requirements of existing codes.

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APPENDIX A - PART I

ANALYSIS OF 55 FT. BEAM UNDER EQUIVALENT STATIC LIVE LOAD

Dead Load

Wt. of Beam = 580 lb/ft

Wt. of bulkhead = 1853 lb

$$M_G = \frac{580(54)^2 x12}{8} + \frac{1853(54)12}{4} = 2,837,000 \text{ in. lb}$$

Equivalent Live Load:

Equivalent static design load = 20,500 lb.*

M_L = 20,500 x 23 x 12 = 5,710,000 in. lb.

Summary of Moments

	Moment (in-lb)
Live Load, M_L	5,710,000
Beam, M_G	2,837,000
Total	8,547,000 in lb.

Prestressing

Area of Steel = 3.68 sq. in.

Initial Pretensioning Force, P_i = 552,000 lb

Final Pretensioning Force (Assuming 20% loss), P_f = 441,600 lb.

*See Fig. 7, p. 15

Stresses

Prestressing

Initial Top Fiber, f_c = +473 psi
Final Top Fiber, f_c = +379 psi
Initial Bottom Fiber f_c = -2458 psi
Final Bottom Fiber, f_c = -1966 psi

Live Load

Top Fiber, $f_c = \frac{M}{S} = \frac{5,710,000}{4790} = -1193$ psi

Bottom Fiber, $f_c = \frac{5,710,000}{4770} = +1198$ psi

Dead Load

Top Fiber, $f_c = \frac{2,837,000}{4790} = -592$ psi

Bottom Fiber, $f_c = \frac{2,837,000}{4770} = +595$ psi

Summary of Stresses (At Midspan)

	Initial 1 Prestress	Final 2 Prestress	3 Dead load	4 Live Load	Initial 5 (1)+(3)	Final 6 (2)+(3)	Initial 7 (1)+(3)+(4)	Final 8 (2)+(3)+(4)
Top	473	379	-592	-1193	-119	-213	-1312	-1406
Bottom	-2458	-1966	598	1198	-1863	-1371	-665	-173

Theoretical Ultimate Moment*

24,525,000 in. lb

Experimental Ultimate Moment

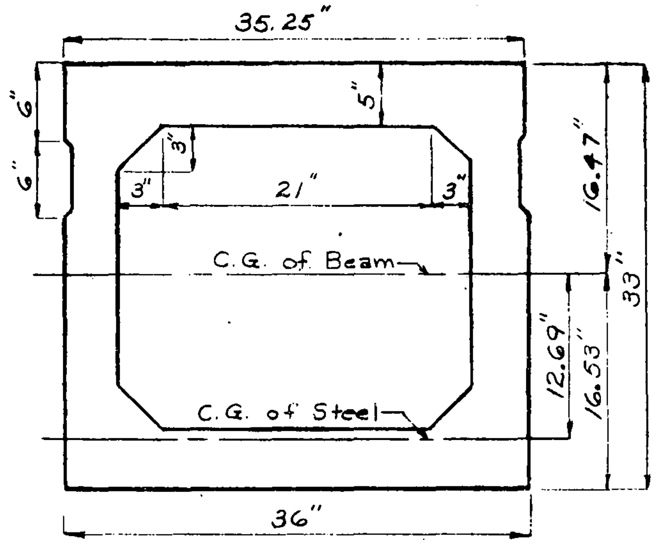
M_{ex} = 78,000(23)(12) + 2,837,000 = 24,365,000 in. lb

* See p. 47 for calculations.

APPENDIX A - PART II

ANALYSIS OF 70 FT. BEAM UNDER TEST CONDITIONS

Section Properties



Center of Gravity of Concrete to Top Fiber, y_t = 16.47 in.

Center of Gravity of Concrete to Bottom Fiber, y_b = 16.53 in.

C. G. of Concrete to C. G. of Steel, e = 12.69 in.

Moment of Inertia of the Section, I_c = 78900 in.^{4}

Section Modulus (top fiber), $S_t = \frac{78900}{16.47} = 4790$ in.^{3}

Section Modulus (bottom fiber), $S_b = \frac{78900}{16.53} = 4770$ in.^{3}

Moments

Dead Load

weight of beam = $558 \times \frac{150}{144} = 580$ lb/ft

wt. of bulkhead = $889.5 \times \frac{150}{144} = 1853$ lb

wearing surface = 90 lb/ft

$$M_b = \frac{580(67.33)^2(12)}{8} + \frac{1853(67.33)(12)}{4} = 4{,}318{,}000 \text{ in-lb}$$

$$M_s = \frac{90(67.33)^2(12)}{8} = 612{,}000 \text{ in-lb}$$

Live Load

$$\text{Fraction of wheel load per beam} = \frac{S}{5} = \frac{3}{5}$$

$$\text{or } \frac{3}{10} \text{ of a lane load}$$

$$\text{From A.A.S.H.O.} - M_{LL} = \frac{3}{10}(937.2)(12) = 3374 \text{ in-kips}$$

$$= 3{,}374{,}000 \text{ in-lb}$$

Shear

Dead Load

$$V_b = \tfrac{1}{2}(580)(67.33) + \tfrac{1}{2}(1853) = 20{,}440 \text{ lb}$$

$$V_s = \tfrac{1}{2}(90)(67.33) = 3030 \text{ lb}$$

Live Load

$$V_{LL} = \tfrac{3}{10}(62000) = 18600 \text{ lb}$$

Summation

	Shear(lb)	Moment(in-lb)
Live Load	18,600	3,374,000
Impact (26%)	4,840	877,000
Beam	20,430	4,318,000
Wear. Surface	3,030	612,000
Total	46,900	9,181,000

Prestressing

Area of Steel = 46(0.08) = 3.68 sq. in.

Initial Pretensioning Force, P_i = 3.68(150,000) = 552,000 lb

Final Pretensioning Force(assuming 20% loss), P_f = 0.80(552,000)

= 441,600 lb

Stresses

Prestressing

Initial Top Fiber, $f_c = -\frac{P}{A} + \frac{Pe}{S}$

$$= -\frac{552,000}{558} + \frac{12.69(552,000)}{4790}$$

$$= +473 \text{ psi}$$

Final Top Fiber, $f_c = -\frac{441,600}{558} + \frac{12.69(441,600)}{4790}$

$$= +379 \text{ psi}$$

Initial Bottom Fiber, $f_c = -\frac{552,000}{558} - \frac{12.69(552,000)}{4770}$

$$= -2458 \text{ psi}$$

Final Bottom Fiber, $f_c = -\frac{441,600}{558} - \frac{12.69(441,600)}{4770}$

$$= -1966 \text{ psi}$$

Live Load plus Impact

Top Fiber, $f_c = \frac{M}{S} = \frac{3,374,000 + 877,000}{4790} = -887$ psi

Bottom Fiber, $f_c = \frac{4,251,000}{4770} = +891$ psi

Dead Load

Wt. Beam: Top Fiber, $f_c = \frac{4,318,000}{4790} = -901$ psi

Bottom Fiber, $f_c = \frac{4,318,000}{4770} = +905$ psi

Surfacing: Top Fiber, $f_c = \frac{612,000}{4790} = -128$ psi

Bottom Fiber, $f_c = \frac{612,000}{4770} = +128$ psi

Summary of Stresses (at center of beam)

	1	2	3	4	5	6	7	8
	P initial	P final	D.L. beam	D.L. w.s.	L.L.+ Imp.	1+3	2+3	4+5+7
Top	+ 473	+ 379	-901	-128	-887	- 428	- 522	-1537
Bottom	-2458	-1966	+905	+128	+891	-1553	-1061	- 42

Ultimate Strength of Section

Plastic Ratio

$$= \frac{1}{1+(f'_c/4000)^2} = 0.334$$

Steel Ratio

$$p = \frac{A_s}{bd} = \frac{3.68}{35.25(29.16)} = 0.00358$$

Neutral Axis Ratio

$$K = \frac{2pf'_s}{(1+\beta)f'_c} = \frac{2(0.00358)(250,000)}{(1+0.334)(5650)} = 0.237$$

Moment Arm Ratio

$$j = 1 - \left[\frac{1+\beta+\beta^2}{3(1+\beta)}\right] K = 1 - (0.362)(0.237) = 0.914$$

Ultimate Moment

$$M = A_s f_s jd = 3.68(250,000)(0.914)(29.16)$$

$$= 24,525,000 \text{ in-lb}$$

Safety Factor

$$S.F. = \frac{24,525,000}{9,181,000} = 2.67$$

Experimental Ultimate Moment

$$M_{ex.} = 74,000(23)(12) + 4,318,000 = 24,742,000 \text{ in-lb}$$

Comparison Between Actual and Theoretical

$$M_{ex.} = 24,742,000$$

$$M_{th.} = 24,525,000$$

$$217,000 = \textbf{Difference}$$

Percent Difference = 0.885%

Principal Tensile Stresses

At Centroid (over support)

$$Q = \left[\frac{(16.53)^2(36)}{2}\right] - \left[\frac{(12.03)^2(27)}{2}\right] + \left[2(3)(3)\tfrac{1}{2}(11.03)\right]$$

$$= 3063 \text{ in.}^3$$

$$S_x = \frac{P}{A} = -\ 989 \text{ psi}$$

$$v = \frac{VQ}{Ib} = \frac{46,900(3063)}{78,900(9)} = 202 \text{ psi}$$

$$S_t = \sqrt{v^2 + (S_x/2)^2} - \frac{S_x}{2}$$

$$= \frac{1}{2}\left[\sqrt{4(202)^2 + (989)^2} - 989\right]$$

$$= + 40 \text{ psi}$$

At 8" From Top (over support)

$$Q = 8(36)(12.47) - \frac{3}{4}(6)(13.47) - 27(3)(9.97) + 3(3)(10.47)$$

$$= 2788 \text{ in.}^3$$

$$v = \frac{46,900(2788)}{78,900(7.5)} = 221 \text{ psi}$$

$$S_x = - 989 + \frac{12.69(441,600)(8.47)}{78,900} = - 387 \text{ psi}$$

$$S_t = \frac{1}{2}(- \sqrt{4(221)^2 + (387)^2} + 387)$$

$$= - 100 \text{ psi} <- 240 \text{ psi}$$

($\therefore$ Minimum stirrups were required)

At 8" From Top (over support at 2.5 x ultimate load)

$$v = 2.5(221) = 553 \text{ psi}$$

$$S_t = \frac{1}{2}(- \sqrt{4(553)^2 + (387)^2} + 387)$$

$$= - 393 \text{ psi} <- 480 \text{ psi}$$

($\therefore$ Minimum stirrups were required)

APPENDIX A - PART III

EQUIVALENT DESIGN LOAD FOR 70 FT BEAM

Introduction

The design was originally intended to be for a 70 ft. center-to-center span, but because of limited space in the Laboratory, this span length could not be tested. It was decided, however, to base the design load on the 70 ft. distance rather than the actual distance of 67 ft. 4 in.. The calculations are given below.

Summary of Moments
(70 Ft. Span)

Live (H20-S16-44)	3550 Kip-In.
Impact	902
Beam	4540
Surface	662
Total (D+L+I)	9654 Kip-In.

Equivalent Beam Loading

<u>Due to Live Load Plus Impact</u>

$$P = \frac{4452}{23 \times 12} = 16.15 \text{ kips per jack.}$$

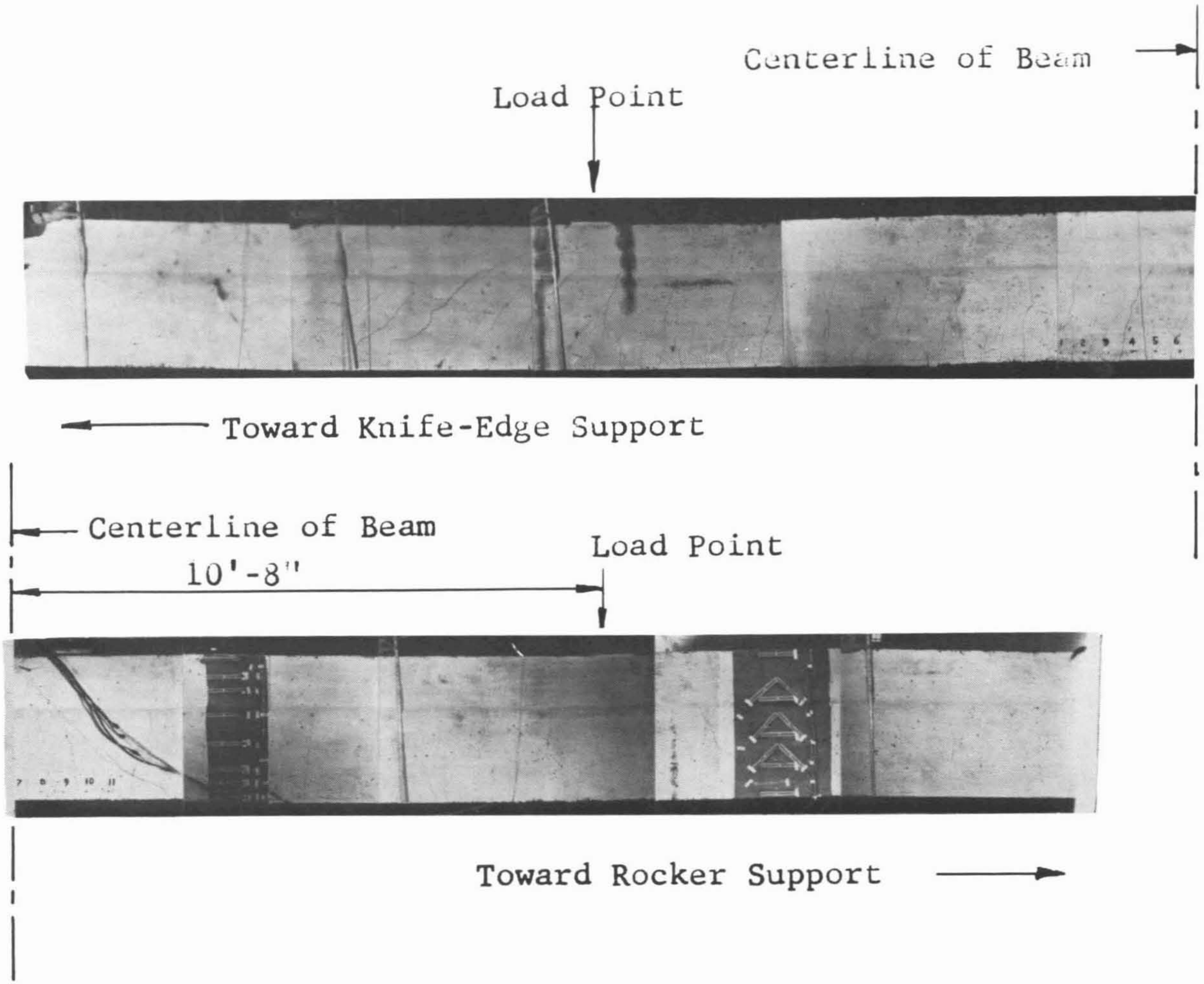


Fig. 21 - Crack Pattern with 55,000 lbs at Each Load Point of 70 ft Beam