Cobourg beam test
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COBOURG BEAM TEST

by

L. J. Marcon

(A co-operative project with the Structural Research Department of the Hydro-Electric Power Commission and prepared for Central Mortgage and Housing Corporation)

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PREFACE

This report contains, in summary form, all the information gained from the most important and most extensive piece of structural research work yet carried out by the Division of Building Research. The delay in the production of the report has been due directly to the time required for the assessment of the large amount of information obtained from the test and its preparation in convenient form for presentation in this report.

The character of the project and its origin is described in the introduction, the test being essentially a full-scale load test almost to destruction of a 100-foot prestressed precast concrete beam continuous over three supports. This report has been prepared for the information of those for whom the task was carried out and others interested in this major investigation. It is hoped that permission may be obtained for the preparation of a series of individual technical papers, which may be published in the scientific and technical press, on certain detail aspects of the test which will make themselves obvious as this general report is studied.

The Division of Building Research are grateful to many who assisted them in the preparation for and the conduct of this project. Information was obtained from many parts of the world on the basis of a preliminary Technical Note regarding the method of testing. As the work developed, the co-operation of the Research Division of the Hydro-Electric Power Commission of Ontario developed to such an extent that the project was regarded as a joint venture. Throughout the work the support and encouragement of the staff of Central Mortgage and Housing Corporation, the consulting and designing engineers, the general contractors and the Department of National Defence assisted greatly and was much appreciated.

Ottawa,
March 1956

Robert F. Legget,
Director.

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COBOURG BEAM TEST

by L. J. Marcon

SUMMARY

This report describes the testing of a 100-foot continuous (two 50-foot spans) I-shaped prestressed concrete beam, prestressed with 56 high tensile steel wires anchored by the Magnel Blaton system. One hundred such prestressed beams constituted the main roof beams for four large Army ordnance warehouses at Cobourg, Ontario.

The test load was applied at the purlin points by eleven 50-ton hydraulic jacks operating against a Bailey bridge reaction frame. The details of the design and construction of the beam, the method of test, the instruments used, the observations recorded and an interpretation of the results are given. The loads applied to the beam were symmetrical and asymmetrical loadings of both spans, the National Building Code test, a 28-day long term load test and the final load of dead load plus 5.5 live load.

The beam was taken to have failed under about six times the design load. The load carrying capacity of the beam's purlin brackets was also determined by testing.

I INTRODUCTION

(a) History and purpose of the project

In 1953 Defence Construction Limited (D.C.L.) constructed for the Department of National Defence (Army) four large ordnance storage warehouses at Cobourg, Ontario. Each warehouse is 500 feet long and 250 feet wide, of single storey construction. They were designed as reinforced concrete structures supported on piles by C.C. Parker, consulting engineers of Hamilton, Ontario. General contractor was the Richard and B.A. Ryan Company of Toronto.

Large roof beams are a prominent part of the design. For these beams the contractor submitted an alternative prestressed concrete beam design which had been prepared by Precompression Company Limited of Montreal. After discussion, this design was accepted by the consulting engineer and D.C.L. and the job proceeded on this basis. Central Mortgage and Housing Corporation, (C.M.H.C.) through their regional office at Toronto supervised and inspected construction for D.C.L.

In view of the innovations presented by the new beam design, C.M.H.C. requested the Division of Building Research, National Research Council, to do a field loading test on one beam.

Prior to the test a technical note was circulated to a number of people interested in the test to inform them of the test and to solicit comments and suggestions from them. Many comments were received and many discussions held, particularly with the research staff of the Hydro-Electric Power Commission of Ontario and of the Structures Section of the Division of Mechanical Engineering of the National Research Council. The entire test project was made a co-operative venture between the Division of Building Research and the Research Division of the H.E.P.C.

(b) Description of main beams and construction procedure

Each beam is made from two 50-foot units, the cross section of which is I-shaped, 3 feet deep through the major part of each span, increasing to 5 feet at the haunched end (Fig. 1). The top and bottom flanges are both 18 inches wide, while the web is 8 inches thick. Purlin brackets at 8-foot 4-inch centres are provided to support 25-foot reinforced concrete purlins, which in turn, carry the lightweight concrete roof slabs.

Rectangular purlins 9 inches by 32 inches frame into the continuous prestressed concrete beams at the column heads to provide longitudinal rigidity. All other purlins are simply supported on the beam brackets and are of a T-shaped cross-section, with a depth of 23 inches, a width at the top flange of 9 inches and a web thickness of 3 inches. The main section properties are listed in Table 1.

The 50-foot units were cast at a central casting bed adjacent to the warehouses. Cable ducts were formed by means of rubber cores (Fig. 1). The placing of concrete and moving of the beams at the casting bed was done by travelling gantry cranes. After the concrete had reached a strength of 4,000 p.s.i. the 50-foot units were partially stressed while on the casting bed by means of 8 straight wires (lowest dotted line in the longitudinal section, Fig. 1), each being 0.276 inches in diameter. This partial prestressing was sufficient to overcome the dead weight, handling and erection stresses.

Before erection two 24-wire cables, 100-feet long were inserted in the longitudinal ducts in one of the 50-foot beams. The 50-foot length of cable protruding from the ducts at the haunched end were looped back and placed on top of the beam. After a pair of 50-foot units had been lifted by a crane on top of the columns, meeting with their haunched ends on one column, the 4-inch gap between the haunched ends was partially grouted in with a high strength, quick setting grout, leaving a U-shaped gap to form a continuous duct for the prestressing wires. The grout mixture used was by weight 2 parts portland cement, 4 parts sand and 1 part of a special proprietory mixture containing fine metallic (iron) aggregate. This special mixture counteracts shrinkage through the oxidation and hydration of the iron particles.

After setting of the grout, the cables on top of the one beam were doubled back and inserted in the vertical slot in the haunch of the other beam and then pulled through the appropriate duct. The beam was then ready to be tensioned. A concrete compressive strength of 5,000 p.s.i. was specified before the final stressing of the continuous beams was commenced. The cables were then tensioned in pairs from one end and anchored by the Magnel Blaton system. The required tension in the wires was checked by a pressure gauge on the jack and also by measuring the elongation of the wires.

To obtain in each cross-section the desired line of action of the prestressing force the lower 100 foot cable was then raised up at the middle support. Four 1-inch diameter pins were placed through the web of the beam to hold the cable in the desired position (Fig. 1).

The cable ducts, which extend to the top of the beam near the centre, were then filled by gravity flow with a liquid grout to protect the wires from corrosion and to obtain bond between the wires and the beam. Initially a sand cement grout of about 1 cubic foot of cement, 1 cubic foot of sand and 5 gallons of water was used, but found unsatisfactory since it did not always fill up the full length of the ducts by gravity, probably due to sand particles plugging up the ducts. The remaining spaces were later filled by a pressure grouting firm with a liquid grout without sand.

For the beams in the remaining warehouse a water-cement grout was used which proved satisfactory. The mixture used was 5 bags cement and 27 gallons of water. For the complete grouting of the 100-foot beam about 30 bags of cement were required. Inspection holes were provided along the cable ducts, both to check grouting and to act as air vents.

(c) Design and load requirements

The beams were designed to carry a roof live load of 40 pounds per square foot applied through purlins at the points shown in Fig. 1. The live load per purlin amounts to 8,320 lb., and the dead load of the roof structure per purlin is 7,780 lb. The 100-foot length of the beam weighs approximately 26 tons (Table 2).

II TESTING PROCEDURE

(a) General

In view of the fact that the beam was made under normal field conditions and not under conditions which could be called "controlled conditions" from a research point of view, it was not considered that the project warranted the extensive instrumentation required for a complete strain investigation. The Division, however, proceeded with a limited program of strain measurements, partly in order to gain experience in this type of work and partly to increase the value of the load and deflection measurements.

The set up used to test the beam is shown in Fig. 2. A reaction beam made up from Bailey bridge units was used. The method of applying load and anchoring the test beam and the reaction beam is shown in Figs. 3 and 5. The centre support was fixed whereas both end supports were on steel rollers as shown in Fig. 2A.

The idea of using dead weights as a means of applying load had been rejected because of the total loads which would have to be handled and the fact that all expenditures for this part of the test procedure would have been completely lost after the test. Further, there was no convenient supply near the job of such concentrated loads of steel ingots in sufficient quantity for the test and the necessary rigging would have been costly and complicated. It was felt that most of the money invested in the Bailey reaction beam and hydraulic jacks was recoverable since this equipment could be used on future tests.

(b) Loads

The test loads were applied by means of eleven 50-ton hydraulic jacks, their capacity being approximately 75 per cent greater than the estimated required capacity at failure. The chosen type of jack is also characterized by a lock nut arrangement which is very helpful for applying sustained loads (Fig. 3). All pump units were equipped with pressure gauges which had been calibrated against jack load in a testing machine (Fig. 4). Load cells using electrical resistance strain gauges were placed under the ends and the centre of the beam to measure the reactions and also to serve as a check of the overall applied load. The load cells were also calibrated before the test (Fig. 5).

Difficulty was experienced in applying small loads such as 4,000 lb. per jack ($\frac{1}{2}$ DL) because of the very low pressures to be used on the large jacks. The gauges had a full scale reading of 10,000 p.s.i. Very good agreement between the jack loads and the reactions was obtained at loads equal to and greater than DL + LL.

(c) Deflections

Deflections were recorded at each purlin point and at both ends, thus giving a total of 13 points from which a deflection curve could be plotted (Fig. 6). Deflection measurements were made by three methods.

The smaller deflections of the beam under relatively light loads were measured by dial indicators reading to 0.001 inch and with a range of 1 inch. Gauge blocks were used to extend the range of dial indicators.

Very fine wires attached to the underside of the beam and passed over a system of pulleys to a central location provided a means for measuring the larger deflections under higher loads. Important advantages of this method were that any progressive deflection under load can be detected immediately and that remote measurement is possible.

Further readings on deflections were made by a precise optical levelling instrument at the ends and centre support of the beam to check on any settling.

To check on any possible rotating of the beam, dial indicators were placed at the outer edges of the beam. Thus any difference in readings would indicate tipping. This was done at the ends and the centre of the beam. In addition two spirit levels were used as a quick check on any tipping at the mid-span locations.

(d) Steel strains

Strains on the prestressing wires were measured by electrical resistance strain gauges. In order to minimize the danger of damaging the gauges as the cable was drawn through the ducts it was decided to place them a short distance from one end. The wires which were gauged are shown in Fig. 10.

(e) Concrete strains

Since more concrete test cylinders had been taken from span B, it was decided to take the majority of the strain readings on this half of the beam.

Strains in the concrete were measured by electrical resistance strain gauges and also by a mechanical extensometer. The positions of the gauges are shown in Fig. 6. As a double check, electrical strain gauges were straddled by points for the 8-inch extensometer.

Some difficulties were experienced in taking readings due to temperature and humidity changes in the test area. Variations in taking mechanical extensometer readings was reduced by always attempting to have the same person take the readings.

A constant record of temperature and humidity was kept during the test.

III PROPERTIES OF THE TEST SPECIMEN

(a) General

The two 50-foot units which made up the 100-foot continuous test beam were cast on dates one month apart. Span A was cast on March 16, 1953, initially tensioned on March 21, and span B on February 16 and March 2 respectively.

HISTORY OF TEST BEAM

Date	Description
February 16, 1953	Span "B" was cast
March 2, 1953	Span "B" initially tensioned
March 16, 1953	Span "A" was cast
March 21, 1953	Span "A" initially tensioned
May 11, 1953	First attempt at final prestressing
May 25, 1953	Final tensioning of beam completed
June 30, 1953	Beginning of test period
August 6, 1953	End of test period

(b) Concrete data

The mix used was 4 bags (87.5 lb. each) of high early strength cement, 592 lb. of sand and 928 lb. of gravel, or a weight ratio of 1:1.67:2.65. The average slump was 1.8 inches. The owner's specifications required a minimum ultimate compressive concrete strength of 5,000 p.s.i. at 7 days. Figure 7 gives the concrete strengths obtained from concrete test cylinders and core samples removed from the end section of span B.

The concrete in span B had a slightly higher compressive strength than span A. The average concrete strength of both spans during the time of testing was approximately 7,000 p.s.i.

The concrete core samples from span B gave rather irregular results. The eleven month test produced strengths from 10,760 p.s.i. to 8,560 whereas the two year old cores gave strengths varying from 9,600 p.s.i. to 5,070 p.s.i. The two low core strengths at two years may be due to the concrete having been weakened by beam load test.

(c) Modulus of elasticity of beam concrete

The conversion of the concrete strain data to equivalent stresses necessitates knowing the value of the modulus of elasticity of the concrete. This value is not constant, but varies depending on several factors, chiefly the magnitude and sign of the stress, age of concrete, and previous loading history of the concrete. After a library research and discussions the most representative value of the modulus of elasticity decided upon was 5.2×10^6 p.s.i. To determine this value of E_c the following four methods were used as a guide.

The first method was the comparison of actual bending strains to the stresses calculated from the bending moment diagram. The average E_c according to these calculations is 5.5×10^6 p.s.i.

The second method was the comparison of the designer's theoretical stress figures with the measured strains at several cross-sections. This method gave an average value of E_c of 6.3×10^6 p.s.i.

The third method was the comparison of the theoretical and measured deflections. To make the designer's deflection curve equal to measured curve would have required an E_c of 3.7×10^6 p.s.i. whereas the designer had used an E_c of 5.01×10^6 p.s.i.

Finally a series of compression cylinders and concrete core samples taken from the beam were tested under different loads. Three concrete cylinders tested at the end of the beam test gave an average E_c of 5.5×10^6 p.s.i. The concrete core samples (2.8 inches in diameter, 5.6 inches high, age 11 months) taken from a section of span B gave an average E_c of 5.0×10^6 p.s.i.

(d) Steel data

The requirements for the prestressing wire were as follows:

minimum yield strength (0.2 per cent offset) 160,000 p.s.i.

minimum ultimate tensile strength 215,000 p.s.i.

The working stresses were not to exceed 128,000 p.s.i.

The wire actually used for construction exceeded these minimum requirements. The actual 0.2 per cent yield strength is in the neighbourhood of 203,500 p.s.i. and the ultimate strength is approximately 229,400 p.s.i. The modulus of elasticity was obtained by testing 8 specimens of wire. Both electrical resistance strain gauges and the testing machine's automatic strain recorder were used to obtain the modulus. Both systems gave an average E_s of 27.8×10^6 p.s.i.

IV PRESTRESSING OF THE BEAM

(a) General

No strain and deflection measurements could be made during the initial partial prestressing of the 50-foot units, because the beams were received for testing after this operation had been completed. It was reported that the tensioning of the lower eight 50-foot wires required an elongation of 3 inches and that the average slip occurring during the wedging operation was 1/8 inch.

The beam was ready for complete tensioning of the 100-foot wires on May 11. On tensioning the wires however, it was found that abnormally high pressures were necessary to obtain the required 6 inch elongation. The condition became so serious that it was decided to completely replace the 100-foot cables. The cause of the irregularity was probably the twisting of the cable in the ducts. The test beam apparently was the only one of the 150 beams tensioned that showed such an irregularity. A new pair of cables was inserted and new electrical resistance gauges had to be installed.

On May 25 the new wires were tensioned with no difficulty. Figure 10 shows the sequence in which the wires were tensioned. The same day the lower cable was raised by a jack and sling arrangement and the location pins were inserted.

(b) Deflections during prestressing

The deflections of the beam during tensioning are shown in Fig. 8. The maximum recorded deflection was 0.15 inch.

During the tensioning operation span A always showed a greater upward deflection than span B. This condition was probably caused by the loss of prestress due to friction along the cable duct, thus resulting in a smaller prestressing force in span B. The difference in deflection could also be attributed to difference in age, since span B was the older span. During the prestressing there was very little movement at the ends and centre support. The centre support lifted about 0.004 inch. Deflection behaviour with time could only be followed for one day, since erection of Bailey test structure made the removal of the dial gauges imperative. Figure 9 shows time deflection curve for two similar points on the beams. The decrease in deflection 23 hours after final tensioning was very small as can be seen in Fig. 9.

(c) Steel strains during tensioning of 100-foot wires

The 100-foot wires were tensioned from the end of span A. The electrical resistance strain gauges on the wires were located about 6 feet from the end of span B.

The procedure followed in measuring the steel strains in the wires during tensioning was to take electrical resistance strain gauge readings in the following order:

(i) prior to tensioning of the gauged wire;

(ii) immediately after tensioning the wire but prior to wedging;

(iii) immediately after the wire was wedged.

In addition readings were taken on all tensioned gauged wires after each sandwich plate or eight wires were completely tensioned. Table 3 gives the actual steel strain readings. The elongation of each wire and amount of slip was measured and recorded during the tensioning of the beam. The required elongation of the 100-foot wires was 6 inches, which is equivalent to a strain of 5,000 micro inches per inch.

In Table 4 it can be seen that the upper wires (3, 7, 11, 19 and 23) showed a decrease in strain during the wedging operation, whereas the lower wires (27, 35 and 47) showed an increase in strain. By actual measurement with a steel rule, all wires showed a decrease in elongation during the operation, varying from ½ to 1/16 inch with an average slip of 1/8 inch (Table 5). No reason can be given for this increase in strain measured by the electrical resistance strain gauges. The increases, however, were small.

Wires 3, 7, 11 and 15 showed a decrease in strain between the time of their individual tensioning and the complete tensioning of the sandwich plate of which they formed a part. This decrease was probably due to a redistribution of strain in the wires making up the sandwich plate.

As Fig. 11 shows, with successive completion of tensioning sandwich plates, the steel strains in the upper wires (3, 7, 11, 15, 19, 23 and 31) increased, not decreased as is usually thought. With the exception of wires 23 and 31, the reason for the increase in strain is that the wires are above or quite close to the centroid of the beam's cross-section. Haunching of the beam increases with further tensioning, resulting in an increase in tension above the centroidal axis.

When the lower 100-foot cable was lifted all gauged wires in this cable showed an increase in strain, whereas the wires in the top cable showed a decrease in strain as expected. All wires showed a decrease in strain 6 hours after complete tensioning of the beam.

Due to the possible wetting of the gauges with grout and/or the temperature effects on the active gauges, erratic readings were obtained so that it was not possible to follow loss of prestress with time. After two weeks the electrical resistance strain gauges became stable and new zeros were taken. The last three values in Fig. 11 show the effect of the grout on the gauges. On the right hand side of Fig. 11 is given the corresponding stresses using a value of 27.8×10^6 p.s.i. for the modulus of elasticity for the steel wires.

(d) <u>Concrete strains during tensioning of the 100-foot wires</u>

Concrete strain readings were taken before, during, and after the tensioning of the 100-foot wires. The concrete strain history of the beam obtained by the mechanical extensometer during tensioning is shown in graph form in Fig. 12. The electrical resistance strain gauges gave readings about 50 per cent smaller than the mechanical 8-inch extensometer. As this was the staff's first experience with electrical resistance strain gauges on concrete and the readings measured were abnormally small, only the mechanical extensometer readings were used in the evaluation of the results. During the actual loading test, however, the electrical and mechanical strain gauge readings were found to show much better agreement (Fig. 23).

The strain patterns for the same sections of spans A and B (28-foot mark) are very similar and serve as a good check on our instrumentation. The lower flange at the 1-foot-6-inch mark gave an irregular pattern, but this is probably because of non-uniform strain near the centre support and also because the gauge points on the lower flange at this section were not readily accessible for reading.

There was a gradual uniform transition in the strain diagrams as more wires were tensioned.

The largest change in concrete strain due to lifting of the lower 100-foot cable was in the top flange at the 28 and 39 foot sections of span B and amounted to 20 micro inches per inch or a stress of 104 p.s.i. using an E_c of 5.2×10^6 p.s.i. Very little change in strain was noticed in the lower flange.

The bottom row of strain diagrams in Fig. 12 also shows some of the theoretical strains obtained from the designer's prestressing stress figures using an E_c of 5.2×10^6 p.s.i. The largest variation occurred at the 10-foot-10-inch section.

The dotted line "C" in Fig. 12 is the total of the designer's theoretical figures for the dead weight of the beam and 100 per cent prestress of the eight 50-foot wires plus the measured strains due to prestressing for forty-eight 100-foot wires. This last total will be used for the discussion of the final strain conditions in this report.

(e) Loss of prestress

From the day of prestressing to the day of the first load application (37 days) the average concrete compression strain increase in the top and bottom flange is 117 micro inches per inch.

This increase in strain due to creep and shrinkage represents a loss of 2.3 per cent of the average steel wire strain of 5,000 micro inches per inch at the time of final prestressing.

The reasons why the percentage loss was so small (2.3 per cent) are probably:

(1) The loss due to shrinkage would be reduced because of the age of the beams at time of prestressing; span A was 70 days old and span B was 98 days old. After their initial curing period at the casting site the test beams were exposed to the hot summer air and sun, and covered by canvas during rainy periods. The beams had, therefore, undergone a relatively large percentage of the final shrinkage.

(2) Some creep had already occurred due to the initial prestressing of the eight 50-foot wires. This initial creep was unknown since test beams were received after the initial prestressing was completed.

The total average increase in concrete strains up to the final day of testing was 166 micro inches per inch, which represents a loss of 3.3 per cent of the initial steel wire strain. This increase in strain is due to continuing shrinkage and creep plus the residual plastic flow of the beam produced by the 28-day test.

V TEST PROGRAM AND LOADING SCHEDULE

In order to study the behaviour of the beam under various loading conditions, the loading program in Tables 6A and 6B was carried out. In this report dead load (DL) always represents the weight of the roof structure excluding the weight of the beam itself. This dead load includes the weight of the purlins, roof slabs and roofing materials. Instruments readings were taken before and after the application of all load increments.

(a) National Building Code test

As there are no provisions in the National Building Code (1941 edition) for tests on prestressed concrete structures the criteria for loading tests on reinforced concrete beams were used. The code specifies that a load of dead load + $1\frac{1}{2}$ live load be left on the beam for 24 hours and that the beam must have at least a 75 per cent deflection recovery in the 24-hour period following the removal of the load. Deflection, steel, and concrete strain readings were taken at regular intervals during and after the load was applied.

(b) Asymmetric loadings

The purpose of this test was to check the behaviour of the beam under various combinations of asymmetrical loading and it also served as an additional means of comparing the behaviour of each individual span under load. The first series of asymmetrical loadings only went up to an unbalanced load of DL + 1 LL so as not to cause any permanent damage to the beam. In the second series of asymmetric loadings the load was increased to DL + 2LL and resulted in cracking of concrete.

(c) Twenty-eight day sustained load test

The purpose of this test was to check the creep of the beam under long-term loading. A load of DL + $1\frac{1}{2}$ LL was used. Strains and deflections were read at regular intervals during the 28-day test and also for several days after the removal of load.

(d) Ultimate load test

In the ultimate load test the beam was subjected to a load of DL + $5\frac{1}{2}$ LL without complete failure. At this point one of the jacks became displaced, because of a broken shackle bolt. This displaced the entire test set-up. The beam sprang back and regained its original position, again without complete failure. The slipping of the jack actually subjected the beam to a severe impact test, for the slipping of the jack resulted in the almost instantaneous unloading from DL + $5\frac{1}{2}$ LL. There was no serious structural failure apparent, although cracks appeared in the top flange at mid-span. These cracks were probably the result of the beam springing upwards.

After careful consideration of all factors, it was decided not to go to the trouble and expense of rearranging the entire test set-up merely for the purpose of breaking the damaged beam. Tests were conducted on the strength of the purlin seats before the beam was finally broken up and removed.

VI DEFLECTION BEHAVIOUR DURING TESTING

(a) Symmetrical loadings

In Fig. 13 are plotted the complete deflection data obtained from the wire and pulley system. This figure gives the deflection pattern up to the last measured readings. Readings were not available for DL + 5.5 LL since the jacks became displaced just as this load was reached. For a more accurate picture of deflection see Fig. 14, which is a plot of deflections up to DL + 3.0 LL obtained from dial gauges. Corrections have been made for movement at the ends and centre supports.

One interesting point is that up to DL + 1.5 LL span A always showed a larger deflection, for DL + 2 LL deflections for spans A and B are almost equal and for loads larger than DL + 2.0 LL span B showed the larger deflection. This behaviour was also quite evident from the cracking pattern. Up to DL + 2.0 LL there was no indication of which span was likely to fail first, but at DL + 2.5 LL and DL + 3.0 LL span B's cracking pattern indicated that it would probably fail first.

Figure 15 is the load deflection curve for the beam. It is a typical curve for a prestressed concrete beam consisting of two portions, the straight line followed by a curved line which becomes progressively steeper. At or about the junction of the two portions of the curve, cracking occurs. During the 28-day test the beam cracked at the top of the centre support under a load of DL + $1\frac{1}{2}$ LL. This crack opened up on reapplying DL + 1 LL. The straight line portion of the curve starts to curve in the DL + 1.0 to 1.5 LL range. The steep slope of the curve at DL + 5.25 LL indicates that the beam was near its ultimate load. The maximum deflection recorded for DL + 5.25 LL was 3.29 inches at gauge 10 on span B.

The amount of movement at the ends and centre support was measured by dial gauges. The centre support always settled during application of load, the amount being about 0.20 inch for loads up to DL + 3 LL and increasing to 0.34 inch at DL + $4\frac{1}{2}$ LL.

The end of span A had a downward movement of about 0.06 inch up to DL + 3.0 LL and then started to lift with increasing load. At DL + $3\frac{1}{2}$ LL the lifting had increased to 0.26 inch and had to be corrected by adjusting the nuts on the tie rods. With increased loadings the upward deflection remained at about 0.30 inch. The end of span B had a constant upward deflection of about 0.03 inch up to DL + 2 LL, increased to 0.09 inch for loads up to DL + 4 LL, and at a load of DL + $4\frac{1}{2}$ LL increased to 0.30 inch and remained at 0.30 until end of test.

The deflection readings obtained from the wire and pulley system and the dial gauge system agreed quite closely, the maximum variation being 5 per cent, with most readings closer to 2 per cent.

The deflections of the beam for the same load increased after the beam had been cracked. This is clearly shown in Fig. 16. For loads less than DL the curves are actually reversed but this is probably due to inaccuracies in applying the low loads resulting from temperature and humidity effects. The increase in deflection with the number of load applications becomes obvious for loads greater than DL + 1 LL. For the two gauge positions shown on the graph, the increase in deflection is approximately 0.03 inch for loads up to DL + $2\frac{1}{2}$ LL and increases to 0.10 inch at DL + $3\frac{1}{2}$ LL.

(b) Comparison of actual and theoretical deflections

In Fig. 7 the calculated and measured deflections for a load of DL + 1 LL have been plotted. The calculated values were obtained from the designer for 100 per cent and 85 per cent prestress.

The maximum calculated deflection at the 32 foot mark (0.155 inch) at 100 per cent prestress was 75 per cent of the actual measured deflection (0.206 inch) (average of spans A and B) and 84 per cent for the calculated deflection at 85 per cent prestress (0.173 inch). At 7 feet from the centre support the calculated deflection at 100 per cent prestress (0.005 inch) was 20 per cent of the actual measured deflection (0.025 inch) and 32 per cent of the calculated deflections at 85 per cent prestress (0.008 inch).

(c) Deflections during N. B. C. test

The deflections of two identical points on spans A and B are plotted in Fig. 18 for the National Building Code test. Irregularities in the deflection curve are due to stress redistributions in the steel cables and the effect of changing temperature and humidity. The increase in deflection during the 24-hour load period was 17 per cent for span A and 9 per cent for span B. The immediate deflection recovery for span A on removing load was 88.8 per cent and increased to 93 per cent in 1.5 hours, the respective percentages for span B are 93.5 per cent and 98.5 per cent. Therefore, the beam passed the required minimum deflection recovery of 75 per cent.

(d) Deflections during the 28-day test

The deflection history of gauges 9 and 10 on span B during the 28-day test is shown in Figs. 19 and 20.

The irregularities in the deflection curve are due to temperature and humidity changes at the test site, strain redistributions in the beam, and the adjusting of the hydraulic jack pressures due to creep in the beam. The instantaneous deflection upon applying DL + $1\frac{1}{2}$ LL was 0.258 inch and increased to 0.384 inch or 49 per cent in 28 days. The immediate deflection on removing load was 0.128 inch or a recovery of 66.6 per cent. During the 5-day recovery period the deflection decreased to 0.095 inch giving a total recovery of 75.2 per cent. Similar deflection patterns were obtained for all other deflection gauges. The time and sequence of rises and falls in the curve are the same as those of the concrete strains during the 28-day test (Figs.27, 28, 29 and 30).

(e) Deflection of beam during asymmetrical loadings

The deflections of the beam under asymmetrical loadings are plotted in Fig. 21. The graph shows that the deflection behaviour of the two spans was very similar. Deflections of the beam under asymmetrical loadings of DL and DL +1 LL are almost exactly the same. The maximum difference occurred in the span loaded with DL during the asymmetrical loadings of DL and DL + 1.5 LL. This difference may be due to difficulty in stabilizing the hydraulic jacks due to uplift of the span.

During the asymmetrical loading of DL and DL + 2.0 LL the maximum downward and upward deflections were 0.511 inch and 0.176 inch respectively. The maximum deflection under a symmetrical load of DL + 2.0 LL was 0.330 inch.

VII CONCRETE AND STEEL STRAIN BEHAVIOUR DURING TESTING

(a) Symmetrical loadings

The concrete strains recorded during the symmetrical loadings are plotted in Fig. 22. These values represent averages of all readings obtained from a particular point.

Strain diagrams at the 10-foot-10-inch section and 39-foot section are plotted up to DL + $3\frac{1}{2}$ LL and DL + 4 LL respectively. Readings were then discontinued because strains had exceeded the range of the mechanical extensometer. The increments in concrete strain at the 10-foot-10-inch section are quite small because of small bending moment at that section near the point of inflection.

The strain diagrams at the 28-foot section of span B (four feet from location of maximum bending moment in span) show the effect of cracking. At DL + 2 LL the lower flange had a compressive strain of 10 micro inches per inch. At

DL + $2\frac{1}{2}$ LL the beam had cracked at this location and therefore gave an extremely large strain reading as shown. Similar behaviour was recorded at the 28-foot section of span A.

At the 1-foot-6-inch section the cracking occurred at DL + 3.0 LL, a loading condition at which according to the strain diagram (Fig. 12, A and B) there should still be compression in the extreme top fibre. The reason for this discrepancy is probably the magnitude of calculated stress due to prestressing of the eight 50-foot wires. Theory assumed uniform stress across the 1-foot-6-inch section, which was only 1 foot, 4 inches from end anchorages of the 50-foot wires. To make the strain diagram agree with the tensile strain in the top fibre which must have been present to produce cracking, the compressive strain in the top flange (dotted line) would have to be reduced considerably so that DL + 3.0 LL creates tension.

All strain diagrams up to a load of DL + 2.0 LL were in compression, DL + 2.5 LL being the first load to produce tension at the two 28-foot sections.

A comparison of strains measured by extensometer and electrical resistance strain gauges is made in Fig. 23.

(b) Designer's theoretical stresses

In Fig. 24 are plotted the designer's theoretical stresses of the beam for various load conditions. Due to the small loss in prestress (2.3 per cent) the theoretical stress diagrams with 100 per cent prestress should be used for comparison. Since the initial concrete strains in the beam were unknown, the best means of comparison is the strain and stress diagrams due to DL + 1 LL (Figs. 22 and 24). This comparison again leads to the difficult problem of the modulus of elasticity of concrete. The E_c necessary to make theoretical stresses agree with the experimental strains varies from 9.0 to 3.8×10^6 p.s.i., with an average E_c of 6.3×10^6 p.s.i. This degree of variation makes difficult an accurate comparison of stresses and strains.

(c) Concrete strains due to asymmetrical loadings

In Figs. 25 and 26 the concrete strains due to asymmetrical loadings are plotted. In Fig. 25 the concrete strains of the span loaded with a constant load of DL are shown, whereas in Fig. 26 the concrete strains from the span loaded with DL plus increments of LL are plotted.

In the span loaded with DL plus increments of LL the first crack due to asymmetrical loading occurred at DL + 2 LL at the 28-foot section of span B, as is shown in the strain diagram. A symmetrical load of DL + 2.5 LL later reopened this crack. As expected the asymmetrical loading produced larger concrete strains than the equivalent symmetrical loadings at the 39-foot and 28-foot sections. At the 28-foot section the asymmetrical loading up to DL + 2.0 LL produced a change in compressive strain of 320 micro inches per inch in the top fibre, whereas the change due to the symmetrical loading of DL + 2.0 LL was 215 micro inches per inch. At the 1-foot-6-inch section the change in concrete tensile strain due to DL + 2.0 LL was 235 and 190 micro inches per inch respectively for symmetrical and asymmetrical loadings.

In the span with a constant load of DL no cracks were observed. The larger changes in strain in this span occurred at the 10-foot-10-inch section, which under symmetrical loading was the location of the least concrete strain changes. The concrete strain changes, A, at the 1-foot-6-inch and 10-foot-10-inch sections increased with loading on the opposite span, whereas the concrete strains at the 28-foot and 39-foot sections decreased. The concrete strain changes due to increasing load at the 39-foot section were too small and irregular to pick up with the strain gauges. All strain diagrams A and B in the dead load span were compressive at the maximum recorded asymmetrical loading of DL and DL + 1.5 LL.

(d) Concrete strains during N. B. C. test

The time-strain curves obtained for the N. B. C. test follow a pattern similar to the time-deflection curve shown in Fig. 18. The average increase in concrete strain after application of the full load was 12.5 per cent whereas the average increase in deflection was 13 per cent. The amount of concrete strain recovery was similar to the deflection behaviour.

(e) Concrete strains during the 28-day test

For the 28-day test the mechanical extensometer readings were solely used due to the well-known zero-drift characteristics inherent in electrical resistance strain gauges with time.

Typical histories of the concrete strains during the 28-day test are shown in Figs. 27, 28, 29 and 30. The figures present strains at the 28 and 1-foot-6-inch sections of span B, the sections closest to the maximum positive and negative bending moments. Strain gauges CM 3, 4, 11 and 12 are in regions of increasing compression with load whereas CM 5, 6, 9 and 10 are in regions of decompression with load. In Table 7 are the percentage changes in concrete strains during and after the 28-day test. The concrete stresses calculated from the strain diagrams in Fig. 22 are also tabulated. The stresses at CM 9 and CM 10 are probably much closer to zero stress as explained previously.

The comparison of strain and deflection percentage changes in Table 7 shows that strain gauges CM 3, 4, 11 and 12 represent the deflection behaviour more closely than the gauges at CM 5, 6, 9 and 10. The reason for this behaviour was probably the higher stresses at CM 3, 4, 11 and 12 which in turn would show larger changes. The strain recovery at CM 5, 6, 9 and 10 was higher probably due to small stress present at the gauges.

(f) Steel strains in wires during loading

The actual initial strain in the steel wires during the loading schedule was not known due to the effect of the grouting operation on the gauges. Zero readings were taken on each wire prior to each loading and readings repeated after application of each load. The difference between these readings for 6 typical wires are plotted in Fig. 31 for the last three load series.

Wires 3, 15 and 19 showed a very irregular strain pattern because of their position above and/or close to the centroid of the beam's section. Wires 31, 35 and 47 followed a linear strain pattern as can be seen in Fig. 31.

The maximum recorded increase in steel strain (in wire 47) was 92×10^{-6} inches per inch at a load of DL + 4.5 LL. This increase represented only 1.9 per cent of the initial steel strain immediately after complete tensioning. Larger increase, however, would have been obtained if steel strain gauges had been located at position of maximum moment.

VIII CONCRETE CRACKING DURING TESTING

(a) Symmetrical loadings

The first visible crack was noticed on the 5th day of the 28-day sustained load test. A hairline crack appeared in the upper part of the vertical joint at the centre support. This crack increased in length from 13 inches (when first noticed) to 21 inches at the end of the 28-day test. This crack reopened on applying DL + 1 LL. The first crack due to positive moment occurred at DL + $2\frac{1}{2}$ LL at the 31-foot-6-inch mark on span B. This crack had already appeared during asymmetrical loading of DL + 2 LL. Cracks also occurred on span B in the lower flange at the 25- and 34-foot marks. No cracking was noticed in span A.

At DL + $2\frac{3}{4}$ LL the lower flange of span A at 32 feet also started to crack. The load of DL + 3 LL increased the length of existing cracks and produced an additional crack at the 28-foot mark in the lower flange of span A. This load also produced further cracking in the top flange near the centre support. Span A had a crack at the 3-foot-6-inch mark 15 inches deep, while span B's crack occurred at the 3-foot mark and was 20 inches deep. Further loadings increased the extent of existing cracks and produced a crack pattern as shown in Fig. 32. The largest crack observed before the jacks were displaced was at the 31-foot-6-inch mark on span B and would probably have been the location of ultimate failure. No visible cracks were noticed near both ends of the beam. At the maximum applied load of DL + 5.5 LL the beam had almost reached its carrying capacity because continuous pumping of the jacks produced very little increase in load and cracking was very advanced.

The sudden unloading of DL + 5.5 LL caused the beam to spring up with such force that cracking occurred in the top flange at the 24, 28, 31.5 and 35 foot positions of span B. This sudden unloading also caused cracks to extend the full depth of the beam at the centre support as is shown in Figs. 33 and 34.

These cracks occurred at the joints formed between the precast beams and the grout used to fill up the 4 inch gap at the centre. The presence of the above cracks was one of the reasons for not continuing the test after the jacks kicked out. Further details of cracks are shown in Fig. 35.

(b) Cracking during asymmetrical loading

An asymmetrical load of DL and DL + 2.0 LL produced cracking in the span loaded with DL + 2.0 LL (span B). The concrete could be heard cracking during the application of this load. The first crack occurred at 28 feet, followed by two other cracks at 25 feet and 31 feet 6 inches. The crack at 28 feet occurred between the two strain gauge points at that location.

No cracking occurred in span A during the asymmetrical loadings.

The above cracks reopened on applying a symmetrical load of DL + 2.5 LL.

IX END AND CENTRE REACTIONS

(a) Symmetrical loadings

The total applied load calculated from the hydraulic jack pressures and the load calculated from the eight load cells varied by about 3 per cent. Greater variations were obtained for loads lower than and including DL because of the difficulty of reading low pressures on a high-pressure gauge and also at the final load of DL + $5\frac{1}{4}$ LL, which may be due to bending in the end load cells.

The centre support carried an average of 70 per cent of the total load, with the remaining 30 per cent divided evenly between the two end reactions. The percentage of total load carried by the centre support was very constant throughout symmetrical loadings, the maximum variation being about 1 per cent. Knowing the end reaction and centre support it was possible to calculate the accurate bending moment diagram for the beam. Figure 36 shows the bending moment diagram.

(b) Asymmetrical loadings

Larger differences occurred between loads calculated from load cells and hydraulic jack pressures during the asymmetrical loadings than the symmetrical loadings. The reason was probably the effect of rotation of the beam at the centre support upon the load cells where there was no roller. In Table 8 are shown the average load and percentages of total load at each reaction for the various load combinations.

During the various load combinations the percentage of total load at the centre support remains roughly constant, with the greatest changes occurring at the end reactions. Percentage increase and decrease of the end reactions is not uniform with increase in load. At the asymmetrical loading of DL and DL + 2 LL the reaction at the end of the dead load span was only 1.6 per cent of total load. Under this load all deflection points in the dead load span showed an upward displacement. It was this load which produced cracking in the span loaded with DL + 2 LL.

X PURLIN BRACKET TESTING

Initially it was planned to load the beam through the purlin brackets rather than on the top flange, as this would represent the actual loading conditions in service. It was later reasoned that if the brackets failed before the beam itself, the damage might have been sufficient to prevent further loading of the beam and the main purpose of the test was to check the behaviour of the beam under load, not the behaviour of the purlins.

The purlins were loaded with the yoke arrangement shown in Fig. 37. Two pairs of purlins in span A were tested. The first pair, 16 feet, 8 inches from the centre support carried a load equivalent to DL + 4.0 LL before any cracking occurred. The concrete in this area had been cracked due to previous test loads.

The purlin brackets at the 32-foot-8-inch section were in a region of uncracked concrete and showed no signs of weakness due to previous test loads. The first crack occurred at DL + 9.8 LL, which resulted in a drop in the jack hydraulic pressure. The purlin brackets were loaded to the capacity of the jack, 58 tons or DL + 13 LL producing the cracking shown in Fig. 38. Further purlin bracket testing was considered unnecessary.

XI GROUT INSPECTION

To be removed from the test area the beam had to be broken into several short sections by a pneumatic chipper and a cutting torch.

No visible slipping of the wires and no visible drop in the beam occurred when the anchor plates at the end of span A were burnt off. This indicates that the bond was sufficient to transfer the complete tensioning force. Examining the ducts at the ends of the cut sections revealed that all the wires were surrounded by grout. However, there was a small narrow space between the grouted cable mass and the concrete edges of the ducts due to grout shrinkage. The maximum width of this space was about 1/16 inch.

XII CONCLUSIONS

The result of testing one beam demonstrated that the methods of design were adequate and on the safe side. It must be remembered that only one of the 100 foot beams were tested. The beam gave a load factor of safety of 1.5 at the first crack at the centre support, and a factor of 2.5 for cracking in the span and more than 5.5 against failure considering the live load only. The factor of safety at the maximum load reached, considering all loads, was

$$\frac{294.5}{88.6 + 26.0} = 2.57$$

294.5 Tons = Total Load of DL + 5.5 LL
88.6 Tons = Total Load of DL + 1 LL
26.0 Tons = Total Dead Weight of 100 foot beam

This is more than ample. The German regulation for prestressed concrete specifies a factor of safety of 1.75.*

According to the strain measurements taken, there were no tensile strains in the concrete beam under design load of DL + LL.

The prestressed beam gave ample warning of failure. In service it would be very doubtful if the crack at the centre support would be visible because of the cast-in-place purlin at this location. The crack that would be probably first noticed occurred at DL + 2.5 LL or 157.1 tons, or slightly more than half of the maximum load applied.

Even at a load of DL + 5.5 LL there was no sign of shear cracks at the supports. The concrete in the end zones which was subjected to high concentration of loading under the distribution plates showed no signs of failure.

The electrical resistance strain gauges on the wires showed that eight of the eleven gauged wires were tensioned higher than the 128,000 p.s.i. stipulated in the specifications. The maximum stress recorded was 156,000 p.s.i. or 121.5 per cent greater than 128,000 p.s.i. This maximum stress is 77 per cent of the average measured yield strength of 203,000 p.s.i. and 68 per cent of the average ultimate tensile strength of 229,400 p.s.i. The average steel strain after final prestressing was 5,000 micro inches per inch or

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* Design specifications for structural member in prestressed concrete - 7th Draft, Jan. 1950, by Prof. Dr. Ing. Hubert Rüsch.

139,000 p.s.i. This average measured stress of 139,000 p.s.i. is 68.5 per cent of the yield point stress and 60.7 per cent of the ultimate tensile stress. The German Code specifies that the maximum allowable tensile stress should not exceed 75 per cent of the yield point or 55 per cent of the ultimate tensile strength. In the above figures a value of E_s of 27.8×10^6 p.s.i. is always used.

The impact resistance properties of the beam were demonstrated by the unexpected sudden unloading of DL + 5.5 LL or 294.5 tons, an occurrence which the beam will never have to undergo in normal service.

Although the maximum test load applied was DL + 5.5 LL, the slope of the load deflection curve (Fig. 15) at DL + 5.25 LL indicates that the beam was very near its ultimate failure load. The final increment of load from DL + 5.25 LL to DL + 5.5 LL was quite difficult to apply because constant pumping produced a large increase in deflection but very little increase in load. Cracking was also very far advanced at DL + 5.5 LL. At DL + 5.5 LL a crack at the 31-foot-6-inch section of span B had advanced into the top flange of the beam. Based on the above observations it can be assumed that the beam could not have supported another 0.5 LL increment of load, making the probable ultimate failure load from DL + 5.5 LL to DL + 6.0 LL.

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TABLE 1

SECTION PROPERTIES OF BEAM AT CONCRETE STRAIN GAUGE LOCATIONS

Distance From Centre Line of Beam	Depth of Beam (In.)	Top of Beam From Centroid (In.)	Bottom of Beam From Centroid (In.)	Moment of Inertia ($In.^4$)	Section Modulus	
					Top $In.^3$	Bottom $In.^3$
0'- 0"	60	31	29	226,200	7300	7800
1'- 6"	58.4	30.05	28.35	203,930	6790	7190
10'-10"	36.0	16.7	19.3	59,490	3560	3080
28'- 0"	36.0	16.83	19.17	60,010	3570	3130
39'- 0"	36.0	17.10	18.90	60,780	3560	3220

TABLE 2

MAGNITUDE OF TEST LOADS

Dead Load (Roof Structure)	37.4 p.s.f.
Live Load (Snow Load)	40.0 p.s.f.

Description of Load	Pounds Per Square Ft.	Load Per Jack (lb.)	Total Load (tons)
Dead Load of Roof Structure	37.4	7,780	42.7
Dead Load + ½ Live Load	57.4	11,940	65.7
DL + 1 LL	77.4	16,100	88.6
DL + 1.5 LL	97.4	20,260	111.4
DL + 2.0 LL	117.4	24,420	134.3
DL + 2.5 LL	137.4	28,580	157.1
DL + 2.75 LL	147.4	31,660	174.1
DL + 3.0 LL	157.4	32,740	180.1
DL + 3.5 LL	177.4	36,900	202.9
DL + 3.75 LL	187.4	38,980	214.4
DL + 4.0 LL	197.4	41,060	225.8
DL + 4.25 LL	207.4	43,140	237.3
DL + 4.5 LL	217.4	45,220	248.7
DL + 4.75 LL	227.4	47,300	260.1
DL + 5.0 LL	237.4	49,380	271.6
DL + 5.25 LL	247.4	51,460	283.0
DL + 5.50 LL	257.4	53,540	294.5

TABLE 3

STEEL STRAINS IN WIRES DURING TENSIONING OF BEAM

Wire Number	Number of Wires Tensioned						After Cable Lifted	Time After Cable Lifted	
	8	16	24	32	40	48		6 hrs.	14 hrs.
3	5473	5499	5578	5601	5610	5608	5523	5501	5548
7	4949	4974	5018	5039	5049	5050	4999	4961	5008
11		5153	5191	5219	5219	5208	5168	5117	5168
15		5236	5285	5316	5303	5301	5270	5232	5277
19			4911	4907	4913	4897	4862	4819	4872
23			4723	4735	4724	4695	4680	4645	4651
27				5000	4935	4916	4966	4946	4977
31				5108	5225	5214	5261	5223	5264
35					4592	4566	4657	4616	4684
37					4580	4585	4668	4614	4668
47						4801	4941	4890	4954

Strains in micro inches per inch.

TABLE 4

STEEL STRAINS BEFORE AND AFTER WEDGING OF WIRES

Wire No.	Steel Strains Prior to Wedging	Steel Strains After Wedging	Difference In Strains
3	5571	5508	-63
7	5091	5002	-89
11	5220	5190	-30
15	5258	5258	0
19	4964	4905	-59
23	4751	4725	-26
27	4950	4998	+48
35	4551	4583	+32
47	4751	4836	+85

All strains in micro inches per inch.

TABLE 5

LOSSES IN STEEL WIRE ELONGATIONS DUE TO WEDGING

Tabulated below are the elongations measured during the tensioning of the forty-eight 100-foot wires. The wires are recorded in couples since wires were tensioned two at a time.

	Elongation				Elongation		
Wire No.	Before Wedging	After Wedging	Loss	Wire No.	Before Wedging	After Wedging	Loss
1 and 2	6-1/4	6-3/16	1/16	25 and 26	6	5-3/4	1/4
3 and 4	6-1/4	6	1/4	27 and 28	6	5-7/8	1/8
5 and 6	6-5/16	6-3/16	1/8	29 and 30	6	5-13/16	3/16
7 and 8	6-5/16	6-1/16	1/4	31 and 32	6	5-13/16	3/16
9 and 10	6-5/16	6-1/4	1/16	33 and 34	5-3/4	5-1/2	1/4
11 and 12	-	-	-	35 and 36	5-7/8	5-5/8	1/4
13 and 14	6-5/16	6-3/16	1/8	37 and 38	6	5-11/16	5/16
15 and 16	6-5/16	6-3/16	1/8	39 and 40	5-3/4	5-1/2	1/4
17 and 18	6-5/16	6-3/16	1/8	41 and 42	6	5-15/16	1/16
19 and 20	6-3/8	6-1/4	1/8	43 and 44	6	5-1/2	1/2
21 and 22	6-1/4	6-1/16	3/16	45 and 46	5-15/16	5-7/8	1/16
23 and 24	6-1/4	6-3/16	1/16	47 and 48	5-3/4	5-11/16	1/16

TABLE 6A

LOADING SCHEDULE

Date	Load Series	Load Number	Load	
			SYMMETRICAL LOADING	
June 30	1	1.0	4,000 lb./purlin	
		1.1	DL of roof structure (Approx. 7,780 lb./purlin)	
June 30	2	2.0	4,000 lb./purlin	
		2.1	DL	
		2.3	DL + 1 LL (8,320 lb./purlin, total 16,000 lb./purlin)	
July 1	3	3.0	4,000 lb./purlin	
		3.1	DL	
		3.2	DL + ½ LL	
		3.3	DL + 1 LL	
July 1	4	4.1	DL	
		4.2	DL + ½ LL	
		4.3	DL + 1 LL	
		4.4	DL + 1½ LL (Nat. Bldg. Code Requirement) Held for 24 hours.	
			ASYMMETRIC LOADING	
			Span A	Span B
July 2	5	5.1	DL	DL
		5.2	DL + ½ LL	DL
		5.3	DL + 1 LL	DL
July 3	6	6.1	DL	DL
		6.2	DL	DL + ½ LL
		6.3	DL	DL + 1 LL
			SYMMETRICAL LOADING	
July 3	7	7.1	DL	
		7.2	DL + ½ LL	
		7.3	DL + 1 LL	
		7.4	DL + 1½ LL - Held for 28 days. Recovery period 5 days.	
August 5	8	8.1	DL	
		8.2	DL + ½ LL	
		8.3	DL + 1 LL	
		8.4	DL + 1½ LL	
		8.5	DL + 2 LL	

TABLE 6B

LOADING SCHEDULE (Cont'd.)

Date	Load Series	Load Number	Load	
			ASYMMETRIC LOADING	
			Span A	Span B
August 5	9	9.1	DL	DL
		9.2	DL + ½ LL	DL
		9.3	DL + 1 LL	DL
		9.4	DL + 1½ LL	DL
August 5	10	10.1	DL	DL
		10.2	DL	DL + ½ LL
		10.3	DL	DL + 1 LL
		10.4	DL	DL + 1½ LL
		10.5	DL	DL + 2 LL
			SYMMETRICAL LOADING	
August 5	11	11.1	DL	
		11.2	DL + ½ LL	
		11.3	DL + 1 LL	
		11.4	DL + 1½ LL	
		11.5	DL + 2 LL	
		11.6	DL + 2½ LL	
August 6	12	12.3	DL + 1 LL	
		12.4	DL + 1½ LL	
		12.5	DL + 2 LL	
		12.6	DL + 2½ LL	
		12.7	DL + 2¾ LL	
		12.8	DL + 3 LL	
August 6	13	13.3	DL + 1 LL	
		13.5	DL + 2 LL	
		13.6	DL + 2½ LL	
		13.8	DL + 3 LL	
		13.9	DL + 3½ LL	
		13.10	DL + 3¾ LL	
August 6	14	14.9	DL + 3½ LL	
		14.10	DL + 3¾ LL	
		14.11	DL + 4 LL	
		14.12	DL + 4¼ LL	
		14.13	DL + 4½ LL	
		14.14	DL + 4¾ LL	
		14.15	DL + 5 LL	
		14.16	DL + 5¼ LL	
		14.17	DL + 5½ LL	

TABLE 7

PERCENTAGE CHANGE IN CONCRETE STRAINS

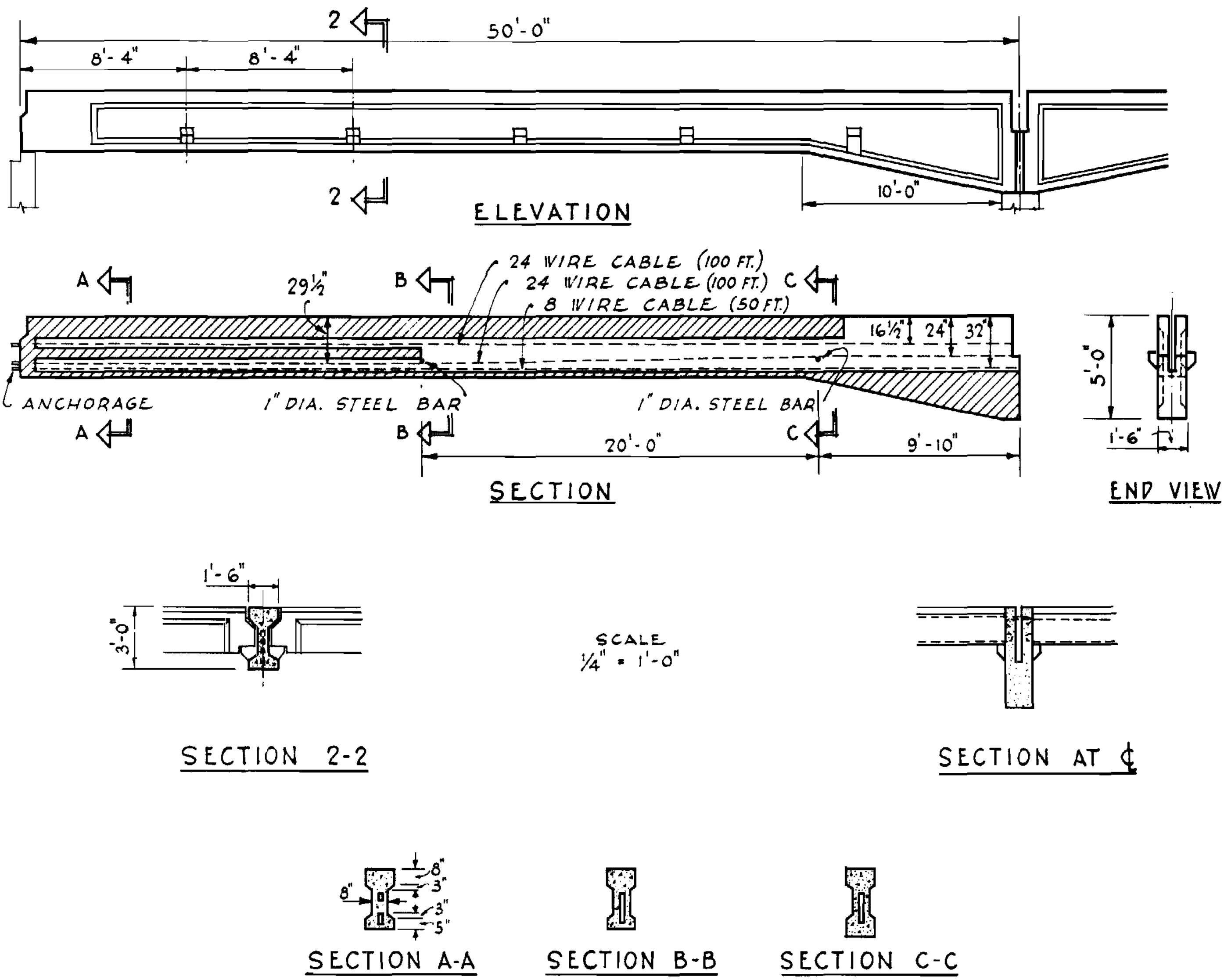
DURING AND AFTER 28-DAY TEST

Gauge No.	Concrete Stress At Gauge p.s.i.	Strain Increase In 28 Days (%)	Immediate Recovery (%)	5 Day Recovery (%)
CM 3	1700	48	64	73
CM 4	1600	48	61	73
CM 5	450	15	84	84
CM 6	360	36	83	83
CM 9	420	45	71	78
CM10	420	25	76	85
CM11	1100	54	63	74
CM12	1100	42	89	83
Total Average		40	73	79
Average 3-4-11-12		48	69	76
Average 56-9-10		29	78	82
Equivalent Deflection Changes		48	67	75

TABLE 8

END AND CENTRE REACTIONS DURING ASYMMETRICAL LOADINGS

Total Load Required (Tons)	Total Load Measured (Tons)	End Reaction		Load On Span	Centre Support		Load On Span	End Reaction	
		Tons	%		Tons	%		Tons	%
54.3	56.0	5.7	10.2	DL	40.3	72.0	DL + 0.5 LL	10.0	17.8
65.7	68.5	5.3	7.7	DL	49.2	71.9	DL + 1.0 LL	14.0	20.4
77.1	78.0	2.7	3.4	DL	55.0	70.5	DL + 1.5 LL	20.3	26.1
88.5	87.5	1.4	1.6	DL	62.1	71.2	DL + 2.0 LL	24.0	27.4



DETAILS of BEAM

FIGURE 1

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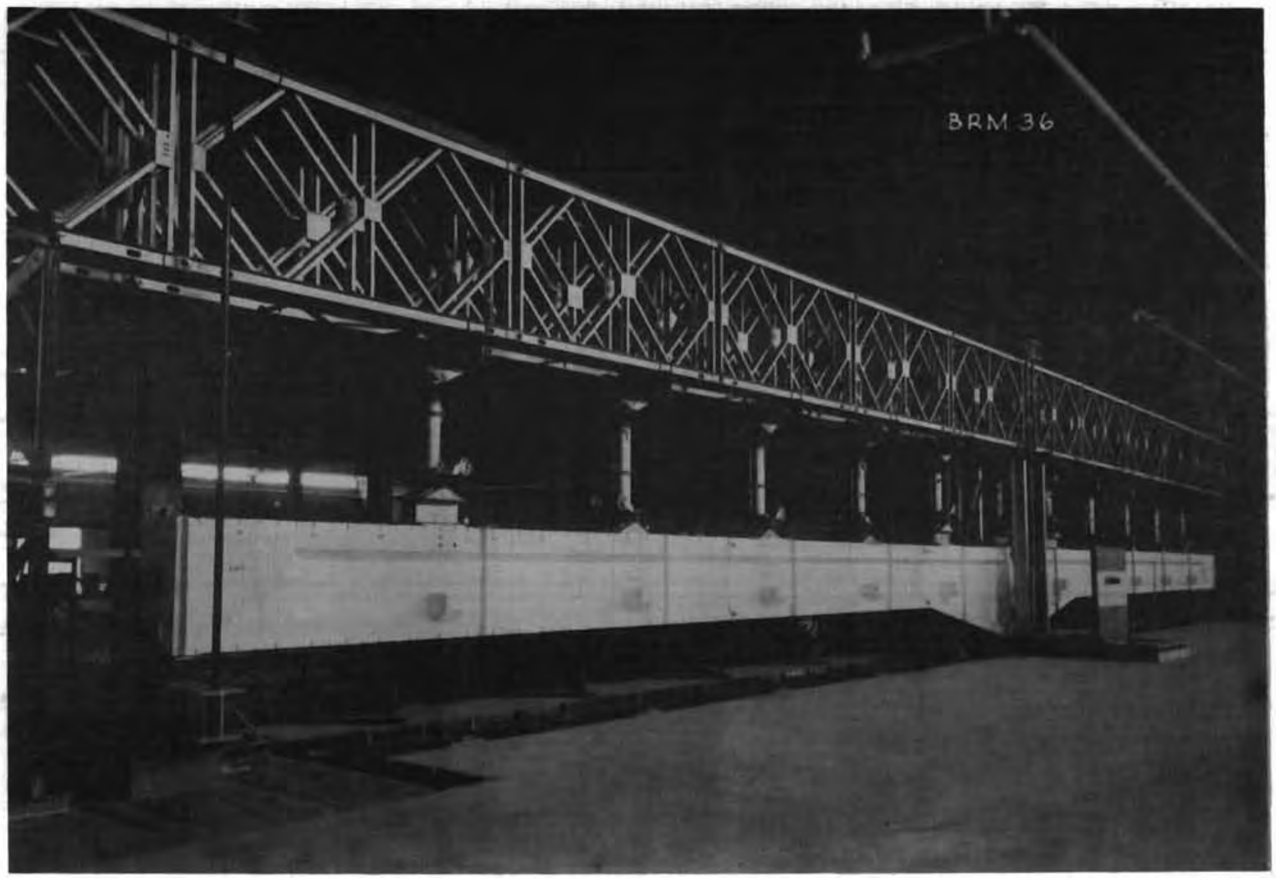
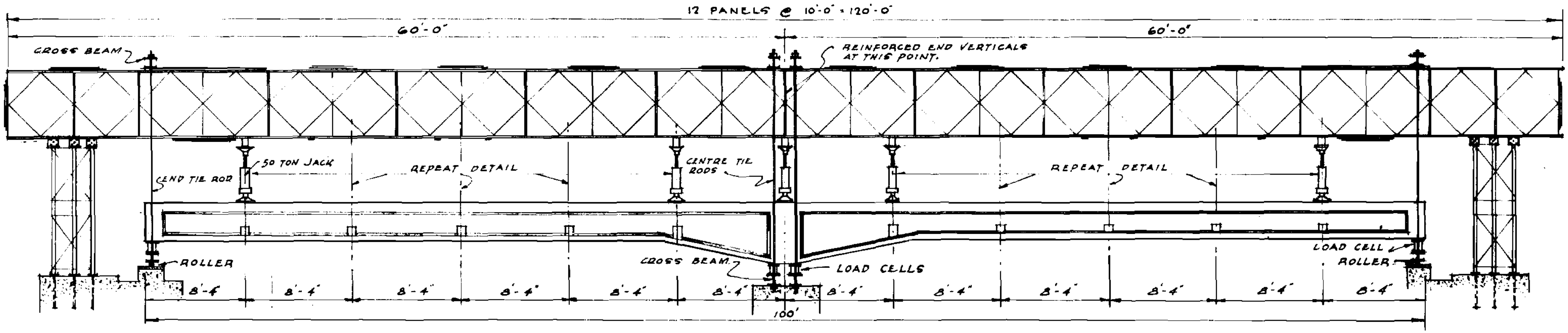


Fig. 2 Over-all View of Test Set Up.



ELEVATION

FIGURE 2A

DBR RPT. 79

BR 439 A
MAY 20 1959 BY T.N.B.

Fig. 3 Typical Hydraulic Ram Used for Applying Load.

Fig. 5 End Reaction of Beam. Note Load Cells and Deflection Apparatus.

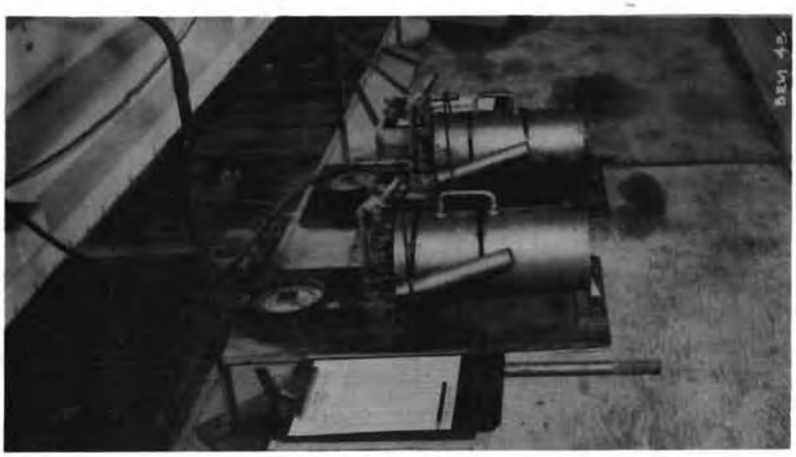


Fig. 4 Hydraulic Pumps and Gauges Used for Loading.

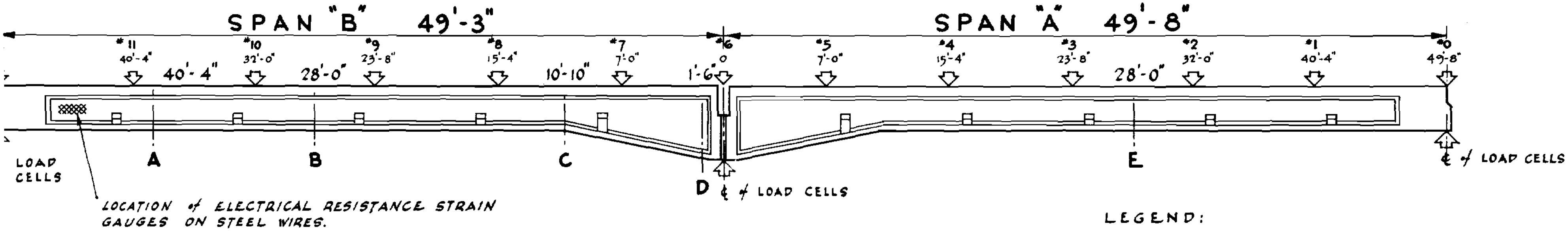


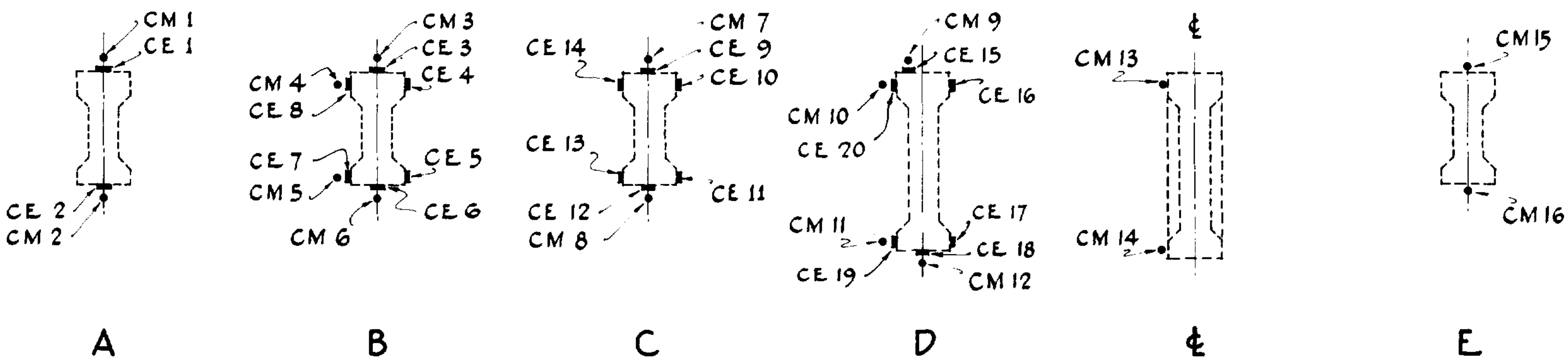
FIGURE 6 LOCATION of STRAIN & DEFLECTION GAUGES

LEGEND:

⇩ LOCATION of DEFLECTION APPARATUS

CE ▮ ELECTRICAL RESISTANCE STRAIN GAUGES ON CONCRETE

CM ● MECHANICAL STRAIN GAUGE ON CONCRETE



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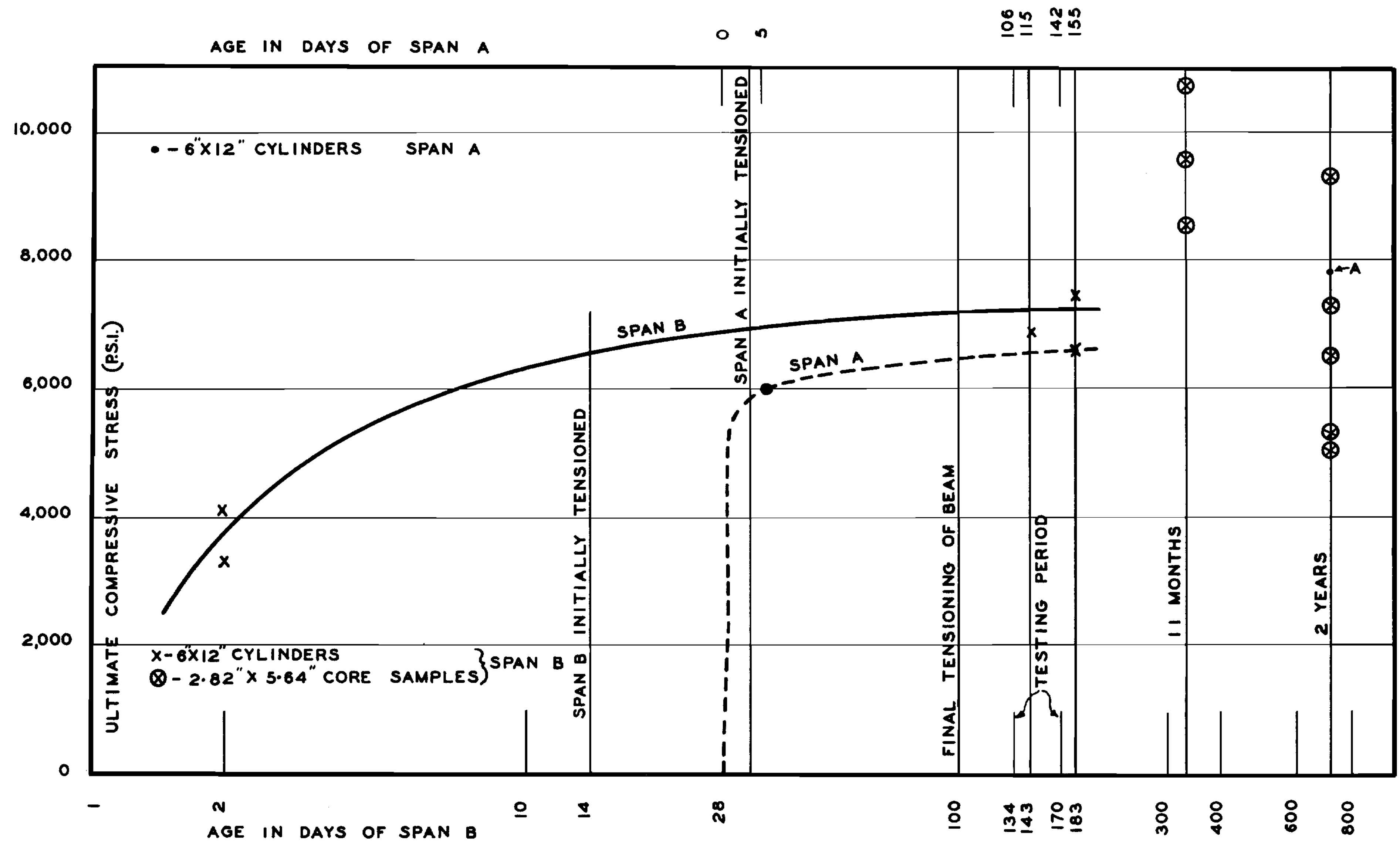

FIGURE 7

TIME - STRENGTH CURVE FOR CONCRETE FOR BEAM

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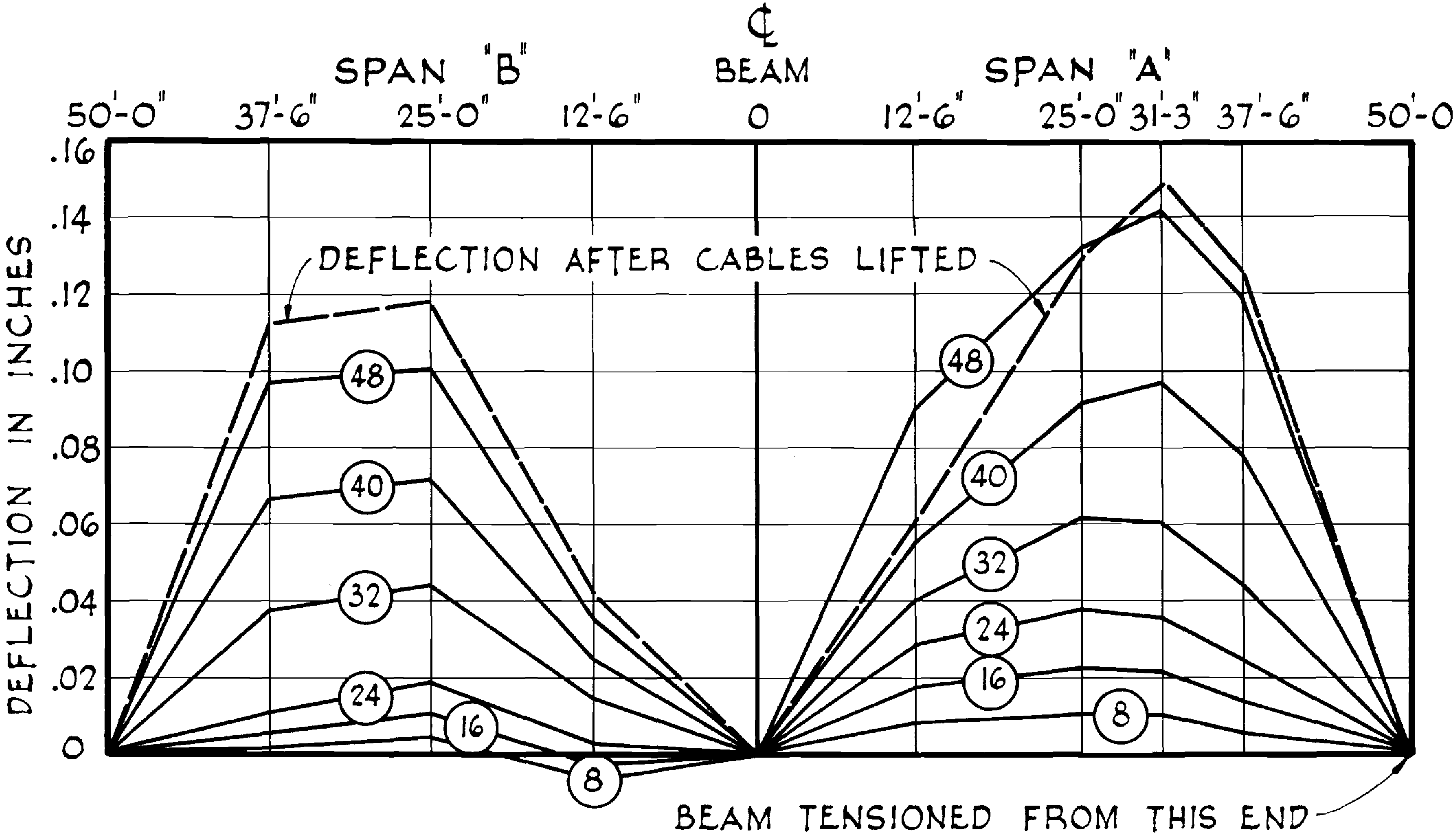


FIGURE 8

HISTORY OF DEFLECTION CHANGES DURING PRESTRESSING OF THE 48-100 FOOT WIRES

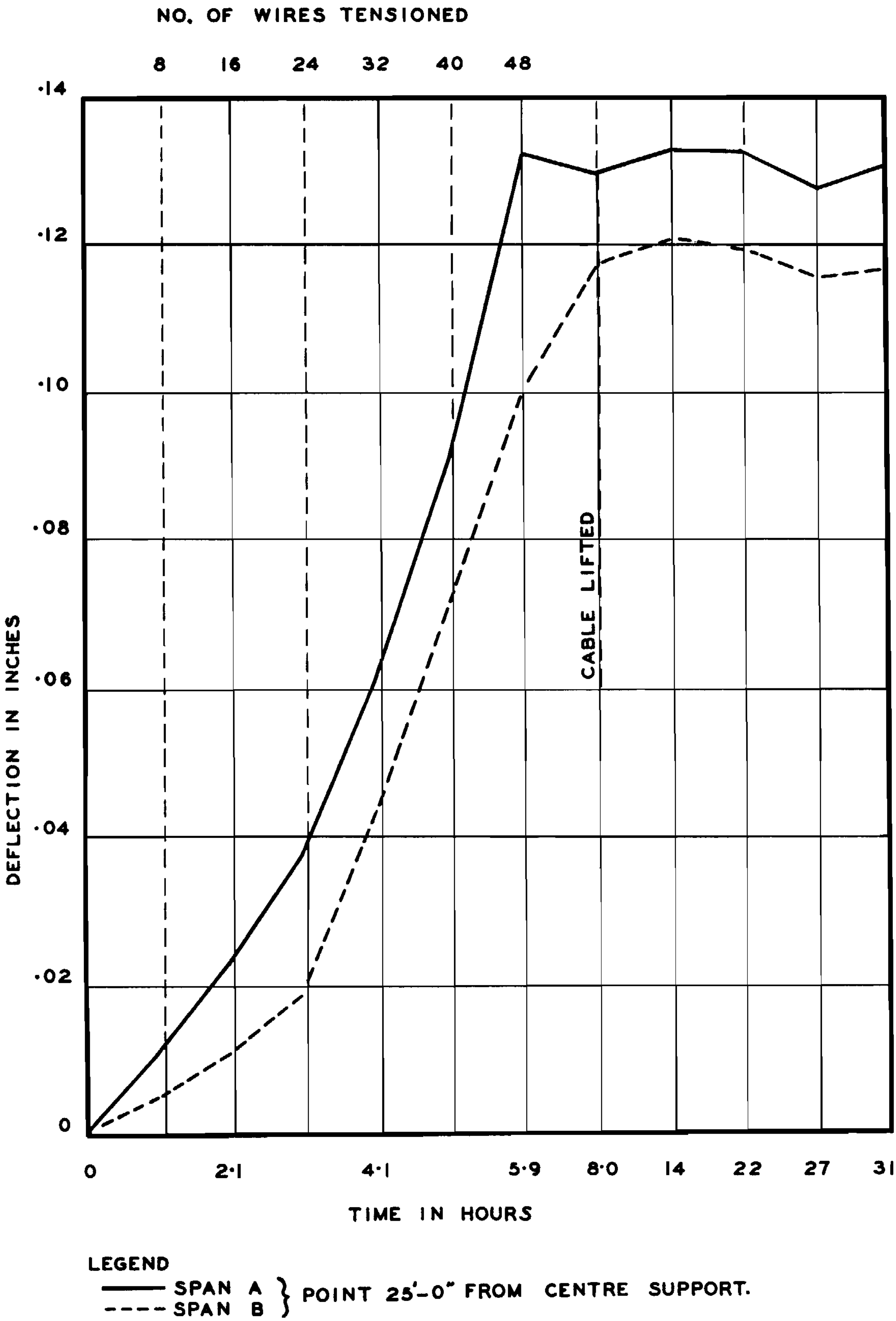
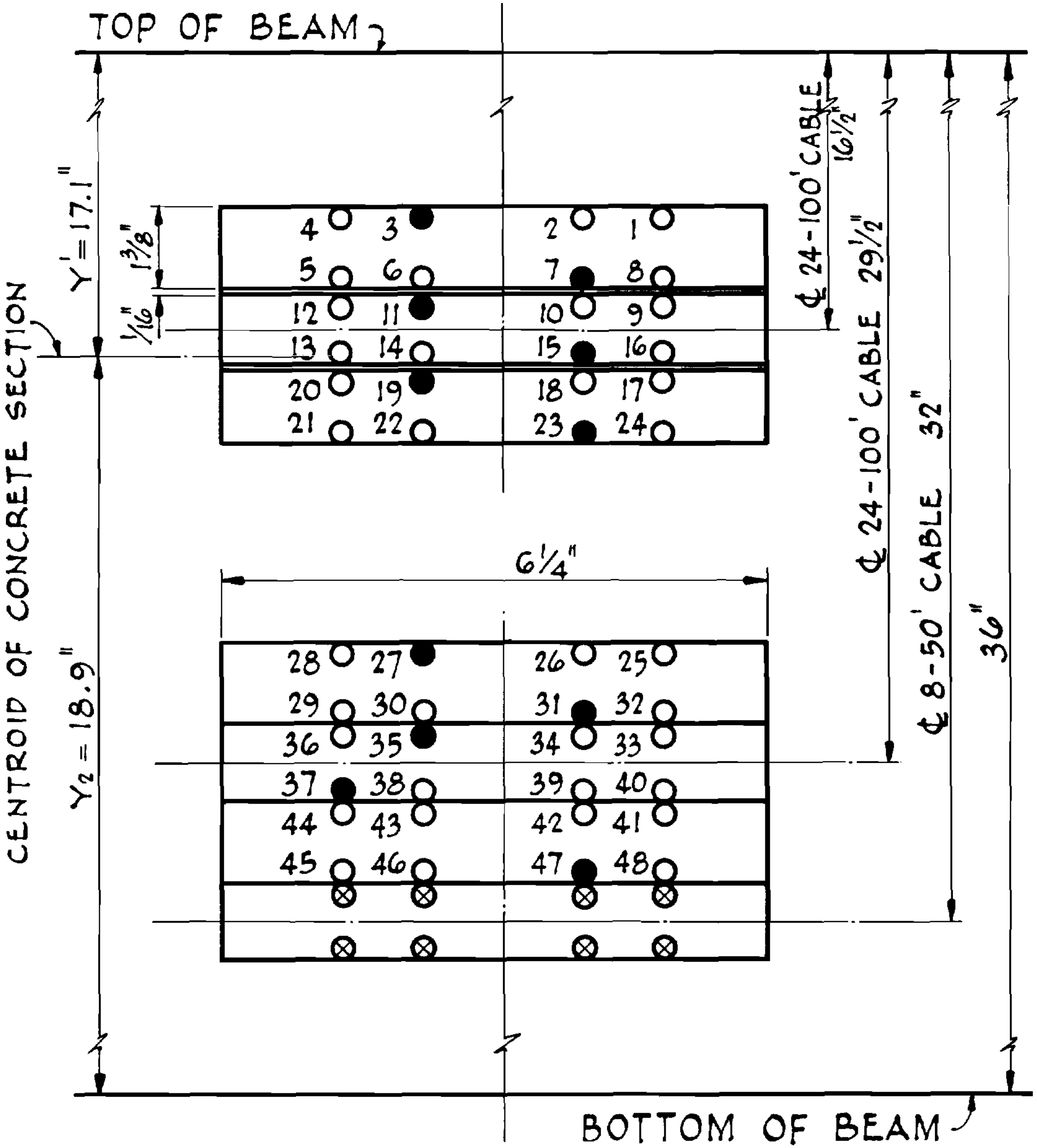


FIGURE 9

DEFLECTION OF TWO SIMILAR POINTS ON SPAN A AND B DURING PRESTRESSING.



LEGEND: ● WIRES WITH ELECTRICAL RESISTANCE STRAIN GAUGES
⊗ 50' WIRES

FIGURE 10
LOCATION of GAUGED PRESTRESSING WIRES

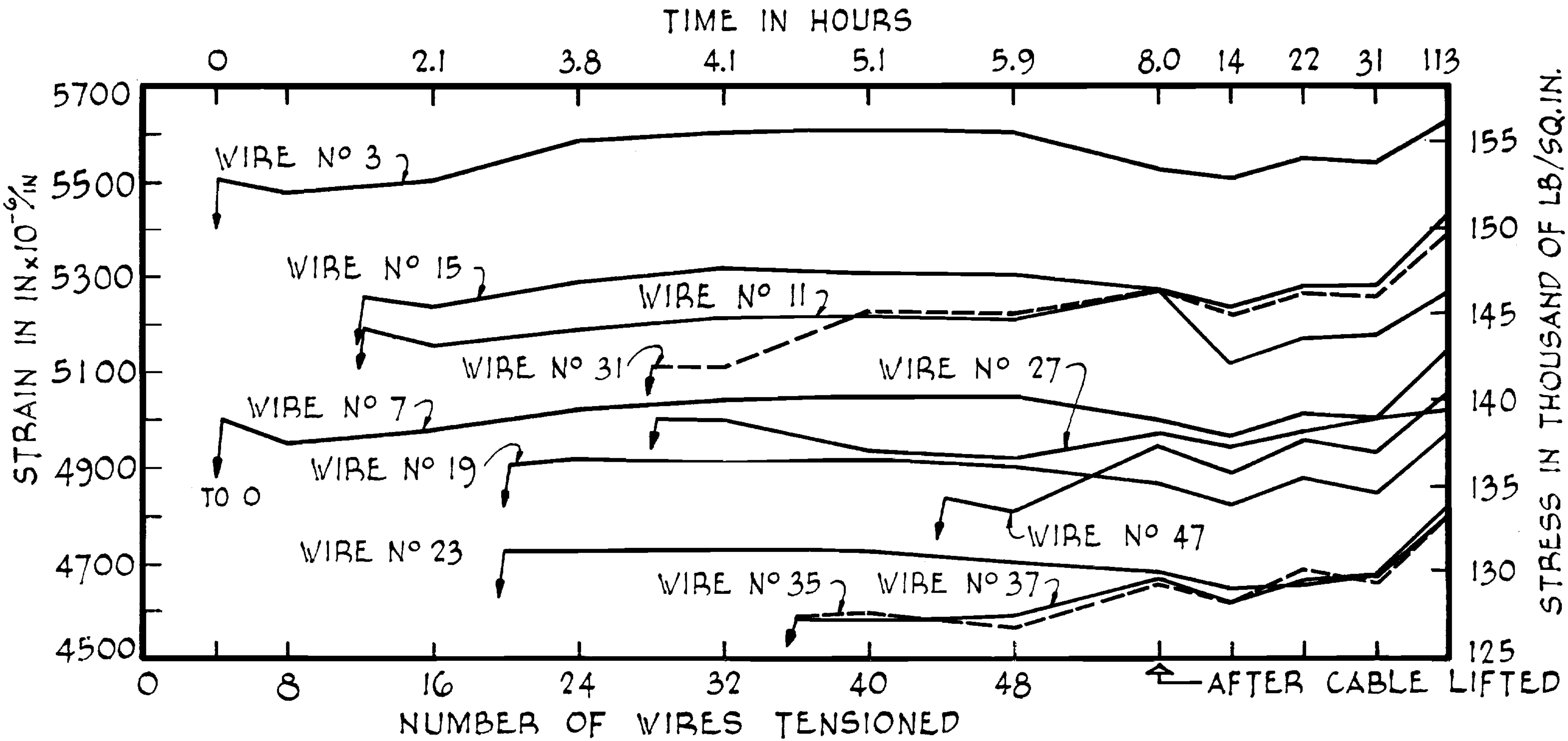
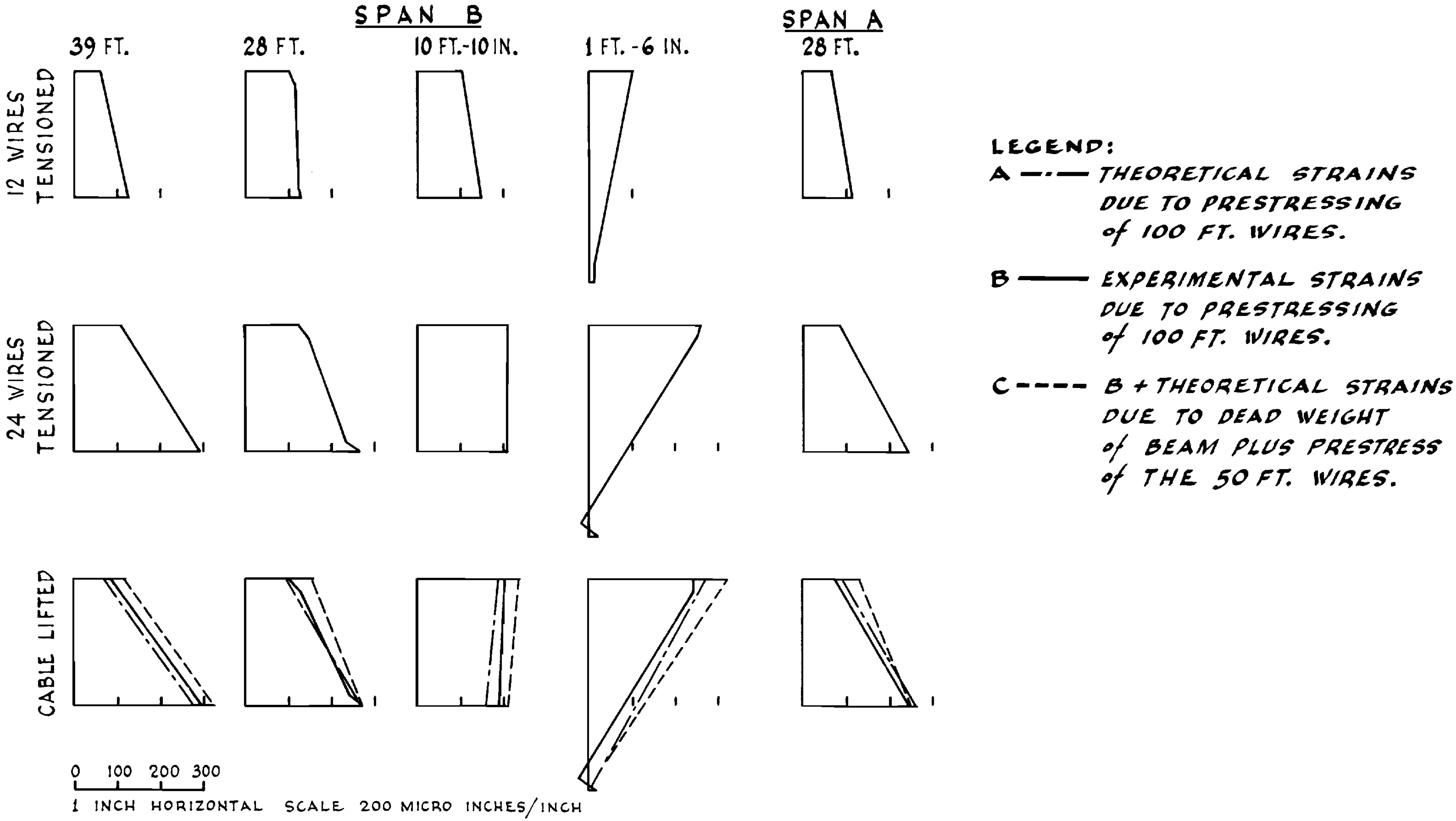


FIGURE 11

STEEL STRAIN HISTORY OF 100 FT. WIRES DURING POST TENSIONING OF BEAM



CONCRETE STRAIN CHANGES DUE TO PRESTRESSING of 100' WIRES

FIGURE 12

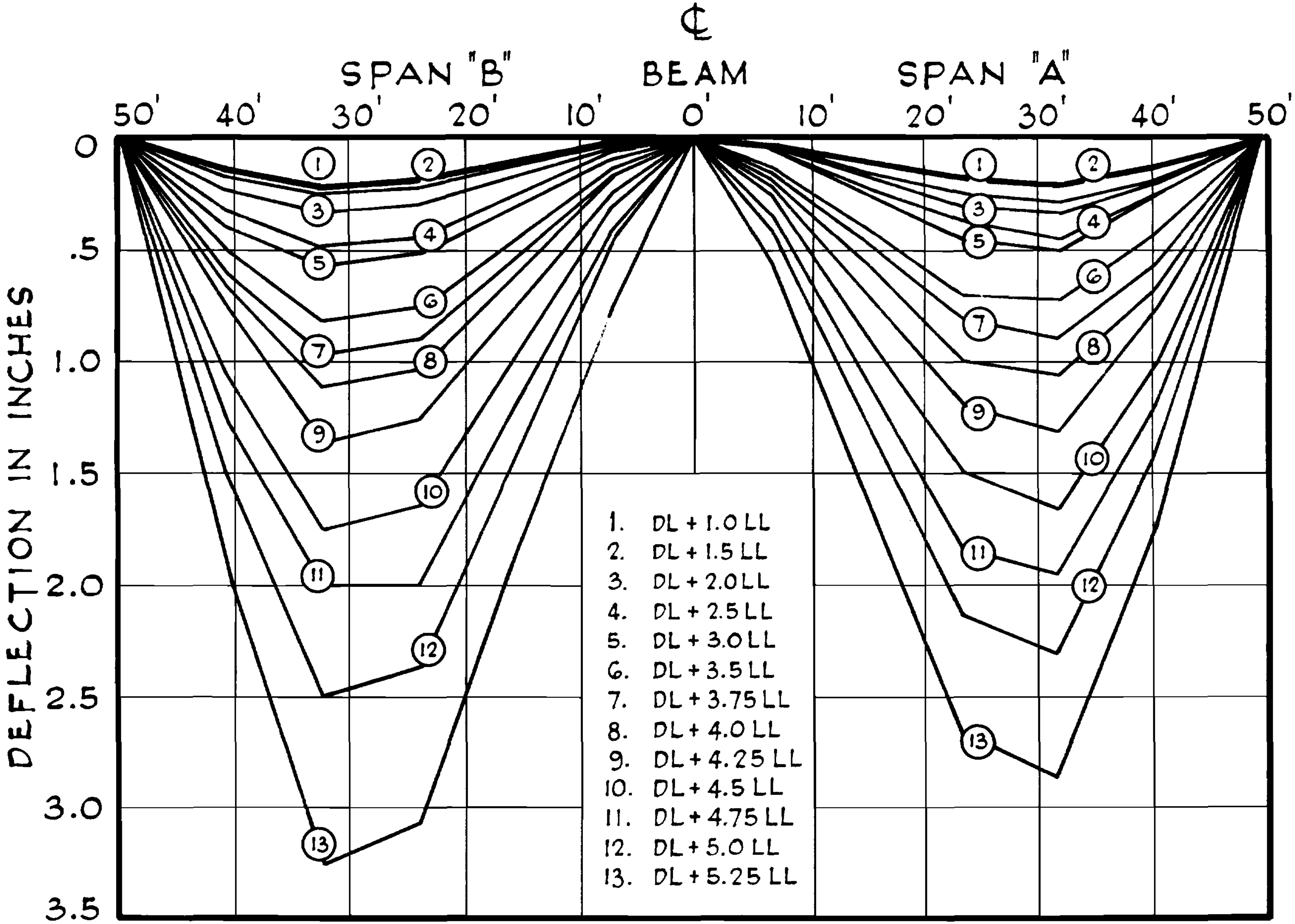


FIGURE 13

DEFLECTIONS OF BEAM UP TO FINAL LOAD

DATA FROM WIRE AND PULLEY SYSTEM

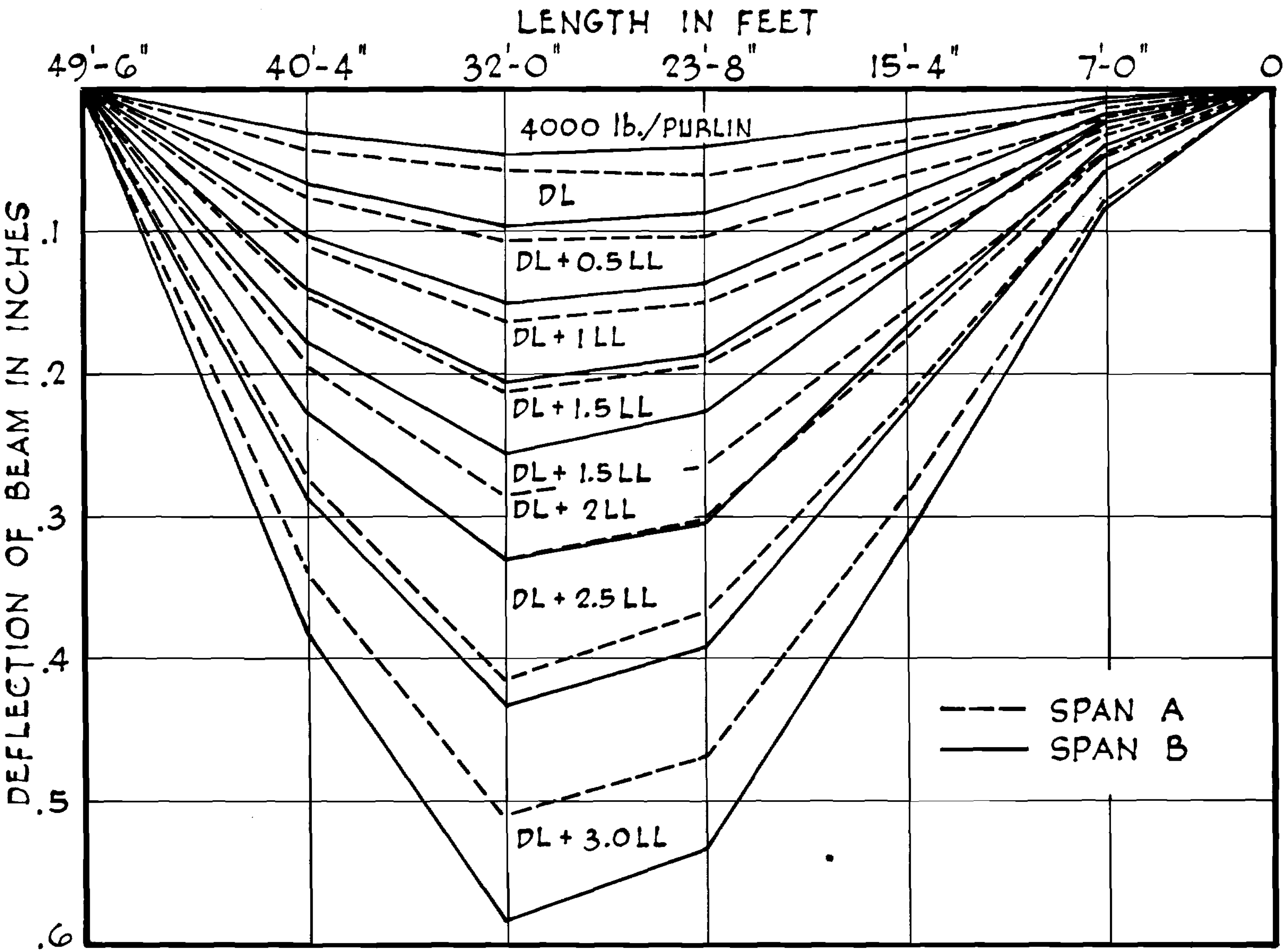
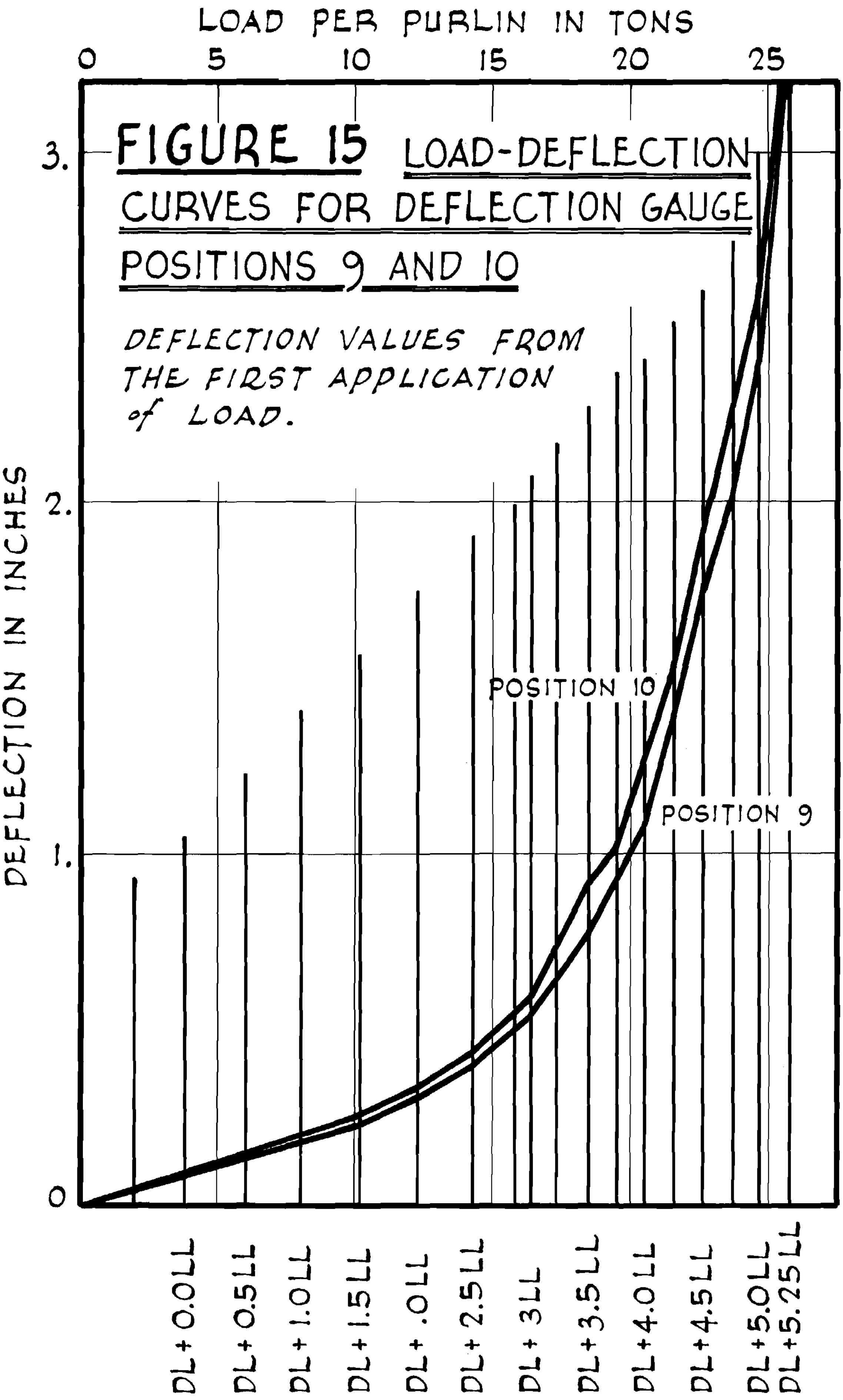
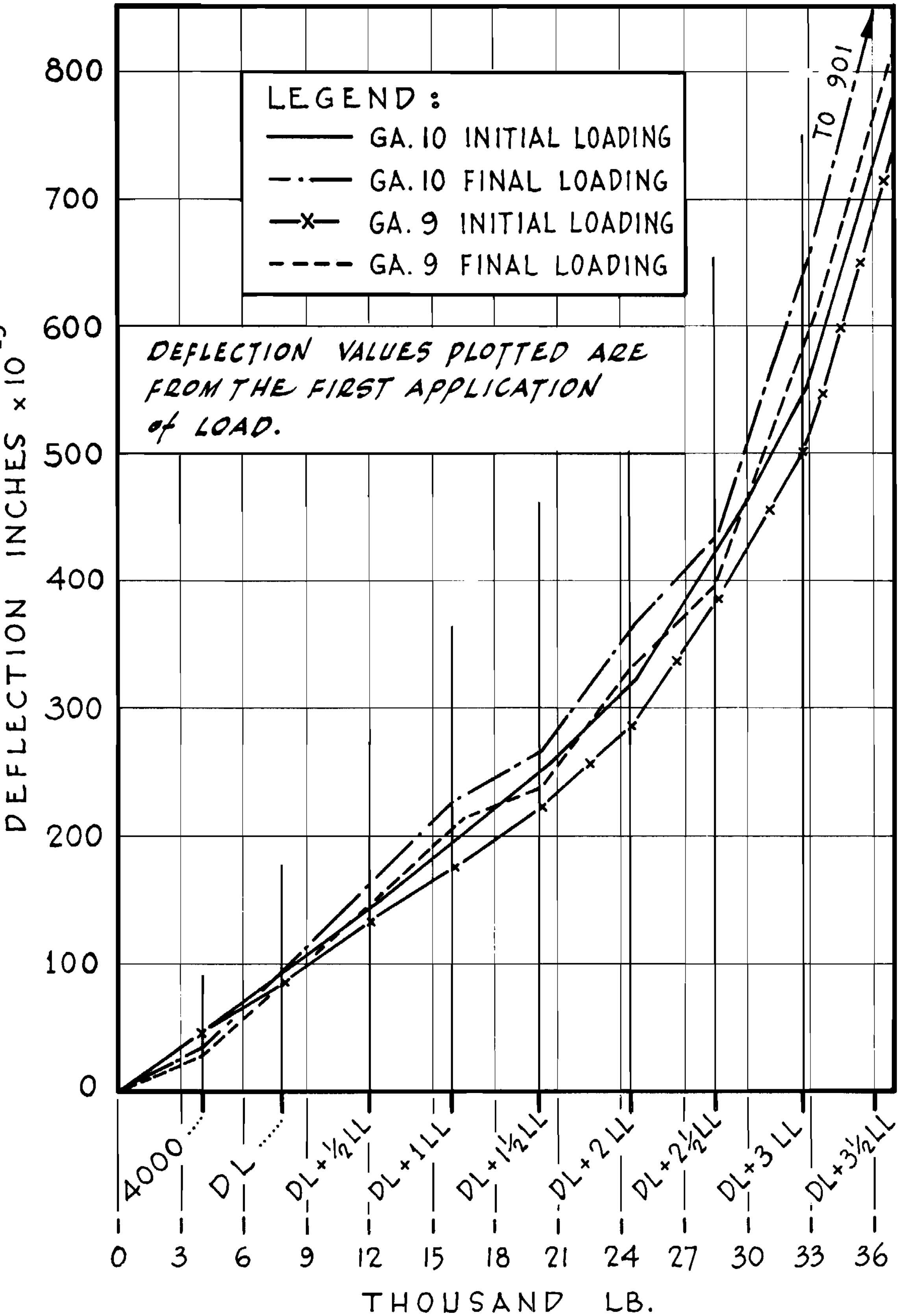


FIGURE 14

SPAN A AND B DEFLECTIONS FROM 0-3 L.L. + D.L.

DATA FROM DIAL GAUGES


LOAD PER PURLIN IN TONS
0
5
10
15
20
25
3.
2.
1.
0
FIGURE 15 LOAD-DEFLECTION CURVES FOR DEFLECTION GAUGE POSITIONS 9 AND 10
DEFLECTION VALUES FROM THE FIRST APPLICATION of LOAD.
POSITION 10
POSITION 9
DEFLECTION IN INCHES
DL+0.0LL
DL+0.5LL
DL+1.0LL
DL+1.5LL
DL+.0LL
DL+2.5LL
DL+3LL
DL+3.5LL
DL+4.0LL
DL+4.5LL
DL+5.0LL
DL+5.25LL



INITIAL & FINAL DEFLECTIONS MEASURED AT VARIOUS APPLIED LOADS

FIGURE 16

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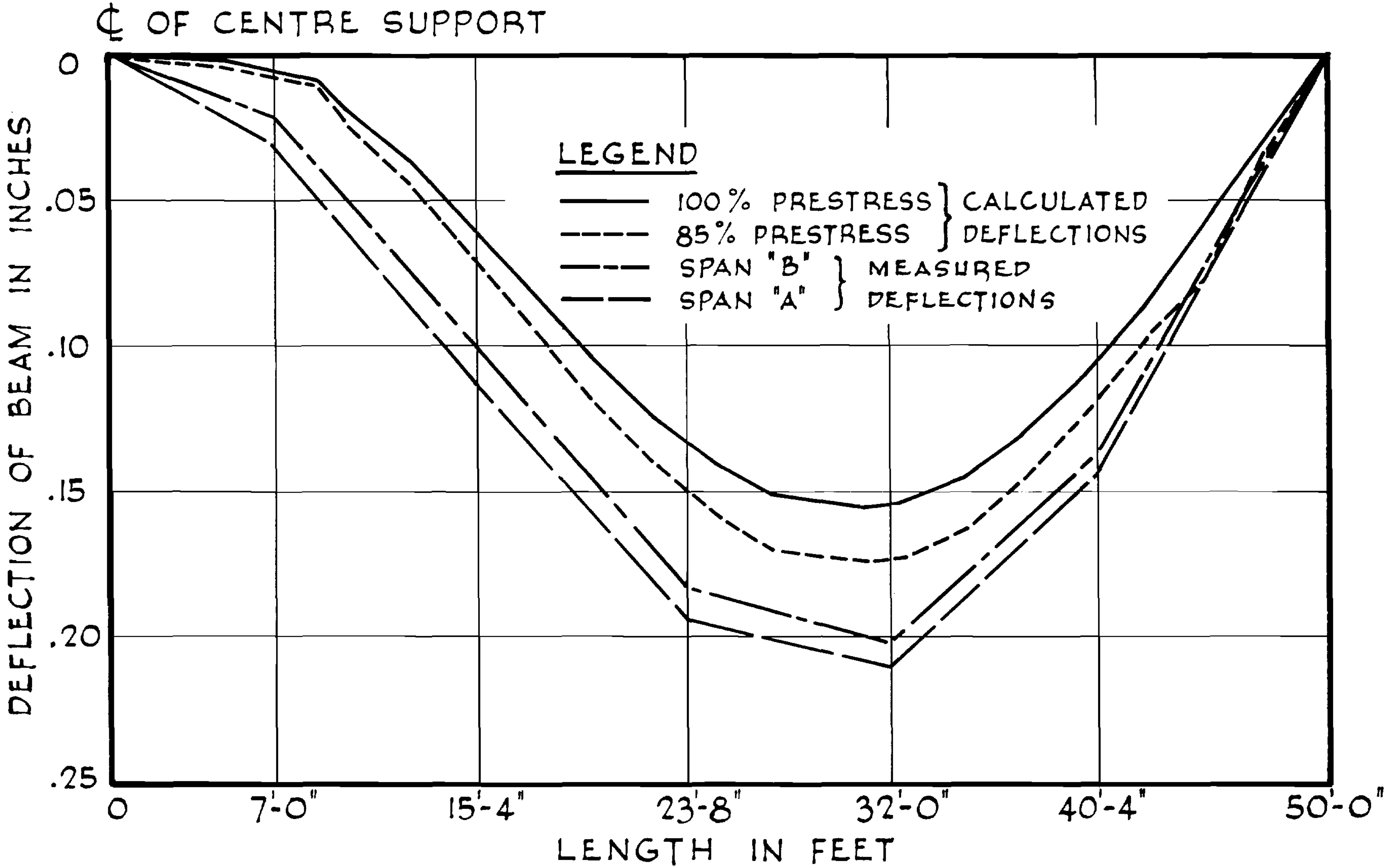

FIGURE 17

CALCULATED AND MEASURED DEFLECTIONS FOR DEAD LOAD PLUS 1 LIVE LOAD

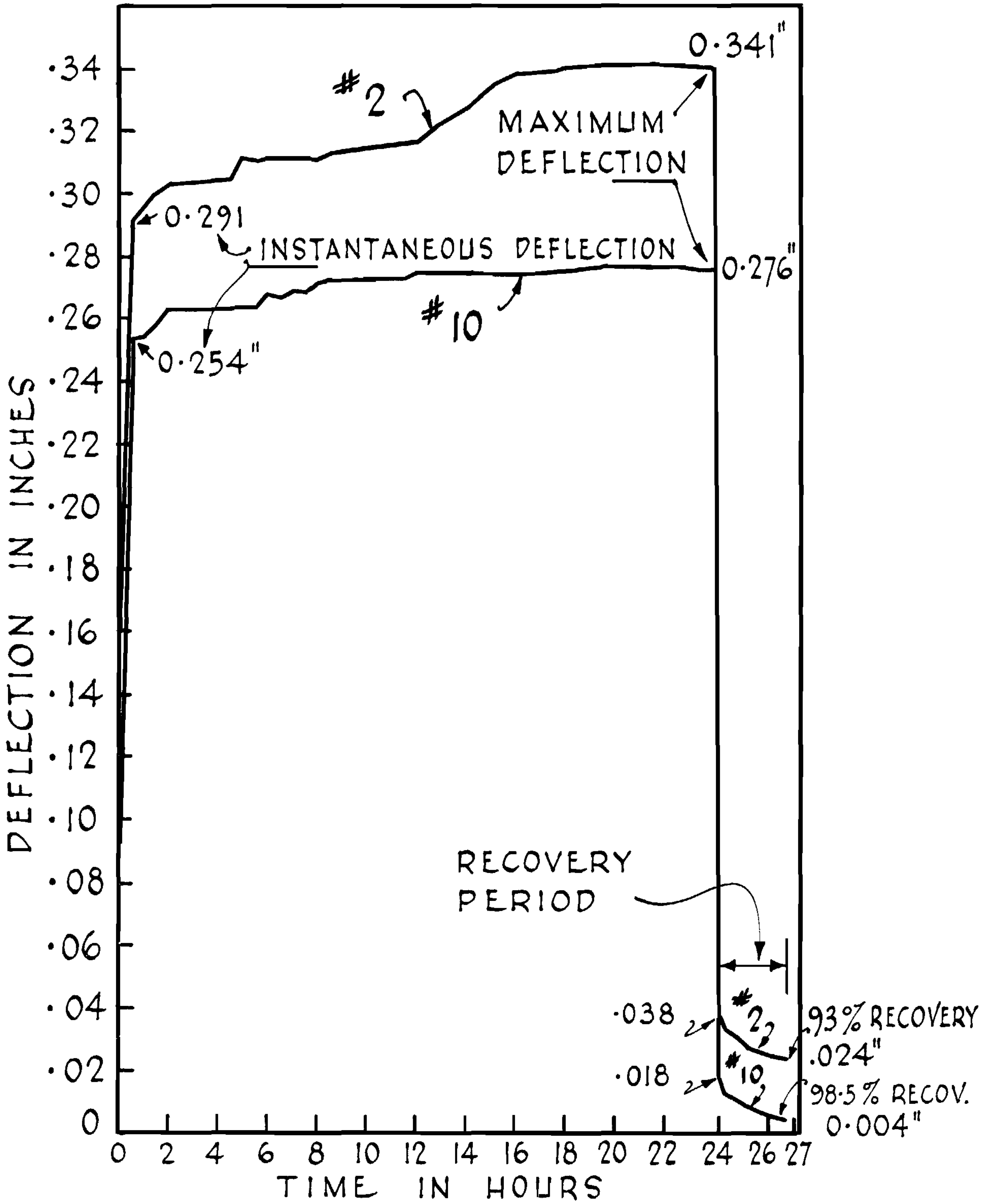

FIGURE 18

DEFLECTION of BEAM AT GAUGES 2 AND 10 DURING NATIONAL BUILDING CODE TEST

(DL + 1½ LL)

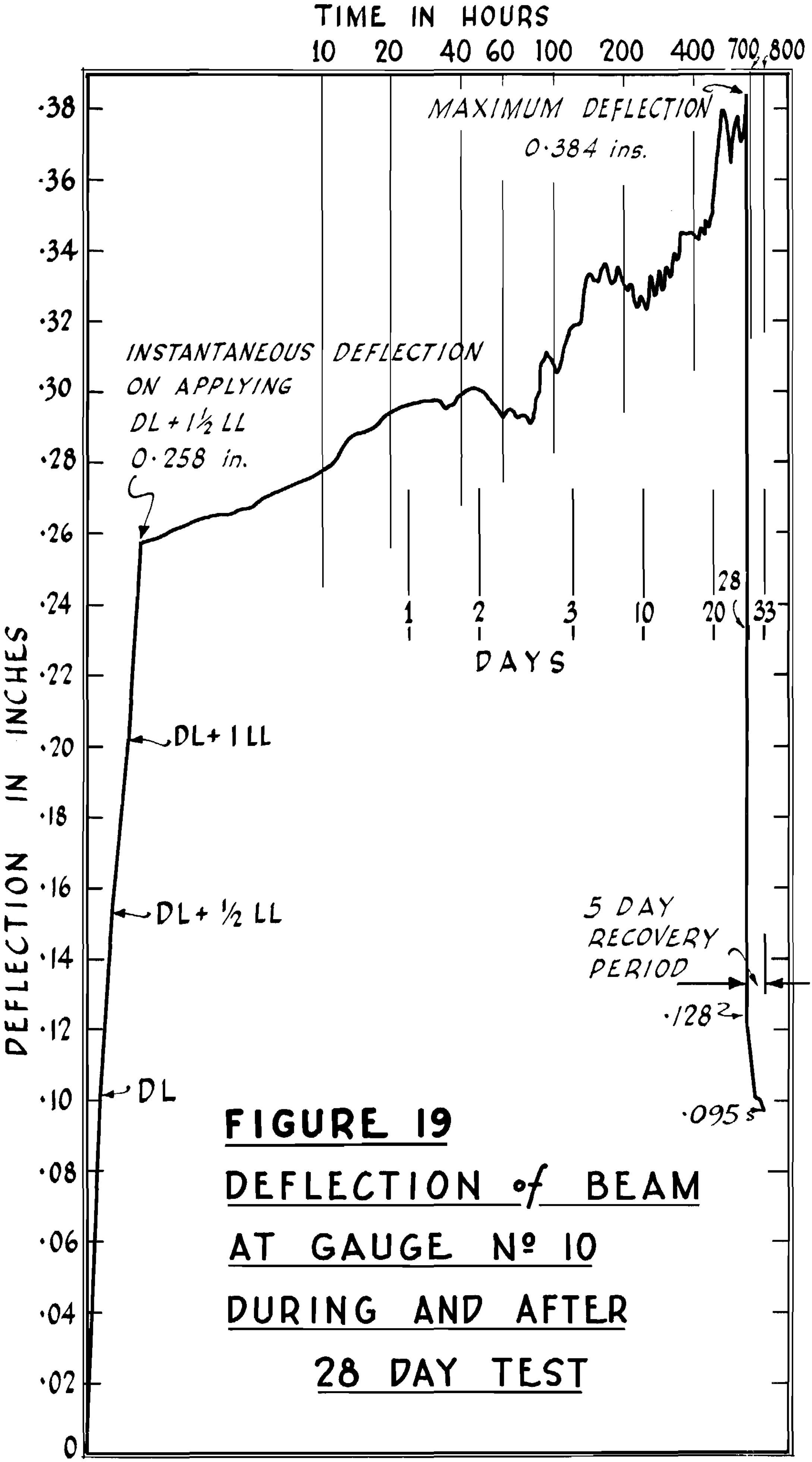

FIGURE 19

DEFLECTION of BEAM AT GAUGE Nº 10 DURING AND AFTER 28 DAY TEST

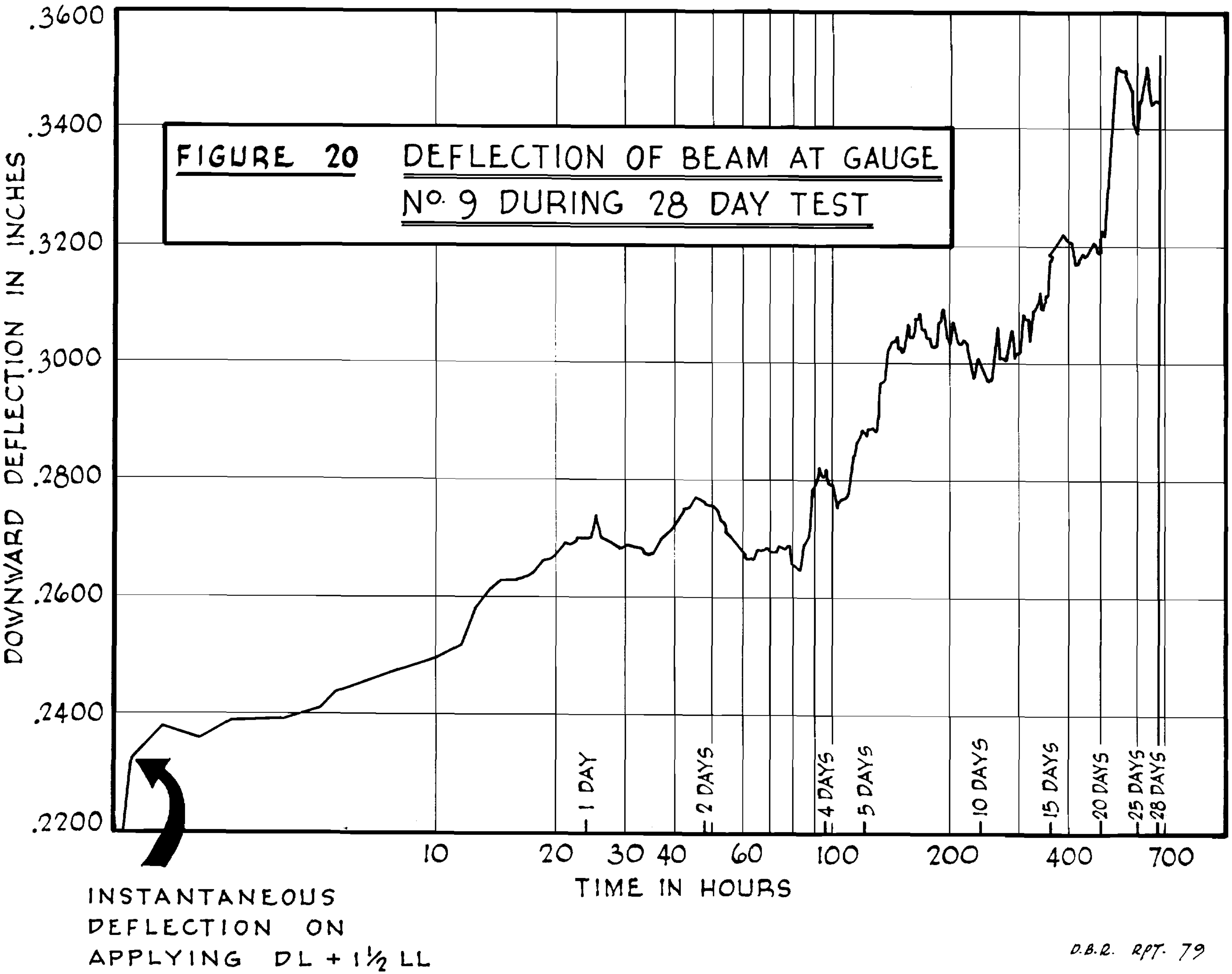

FIGURE 20 DEFLECTION OF BEAM AT GAUGE
No. 9 DURING 28 DAY TEST
DOWNWARD DEFLECTION IN INCHES
.3600
.3400
.3200
.3000
.2800
.2600
.2400
.2200
1 DAY
2 DAYS
4 DAYS
5 DAYS
10 DAYS
15 DAYS
20 DAYS
25 DAYS
28 DAYS
10
20
30
40
60
100
200
400
700
TIME IN HOURS
INSTANTANEOUS
DEFLECTION ON
APPLYING DL + 1½ LL

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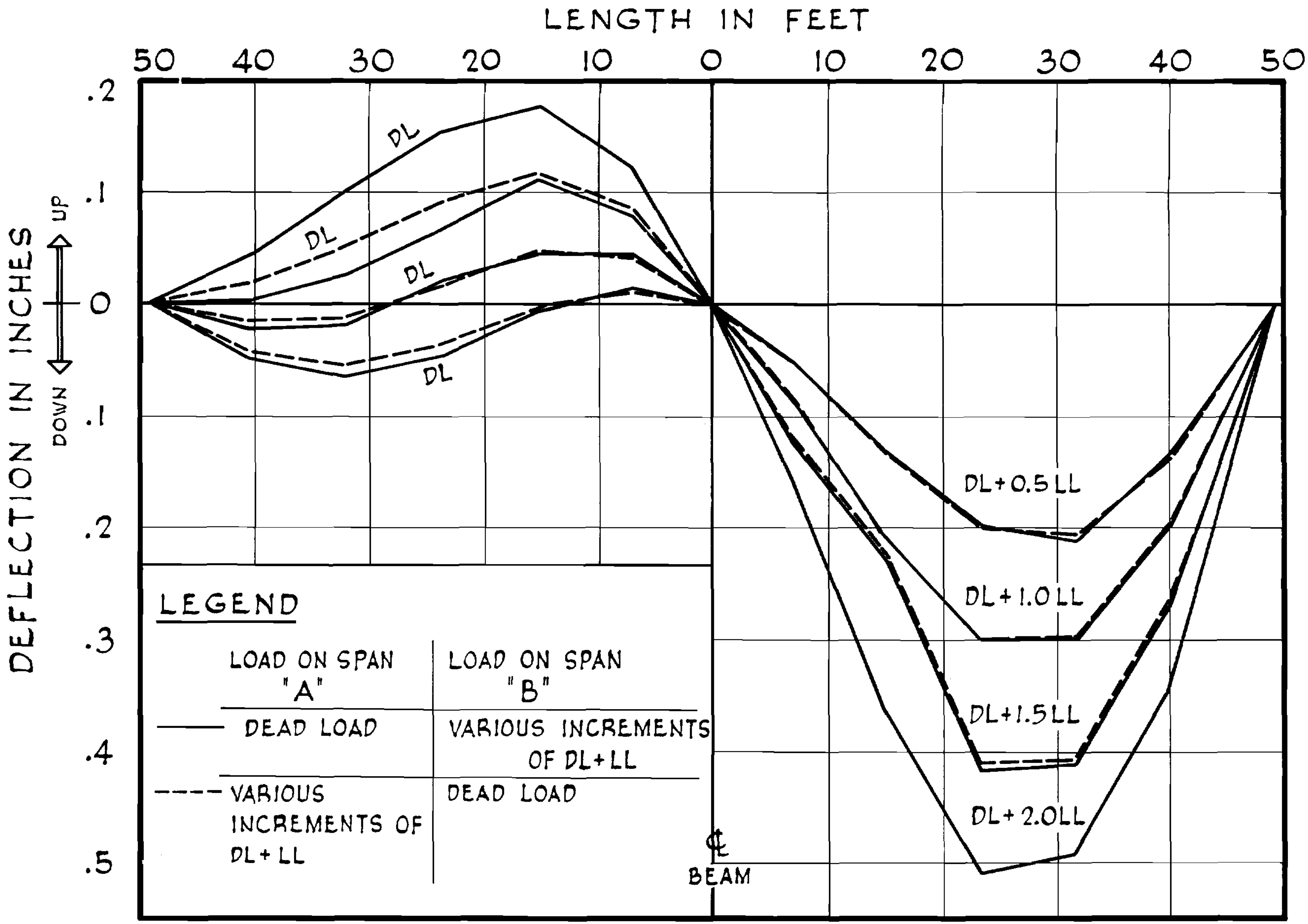


FIGURE 21

DEFLECTIONS OF BEAM DURING ASYMMETRICAL LOADING

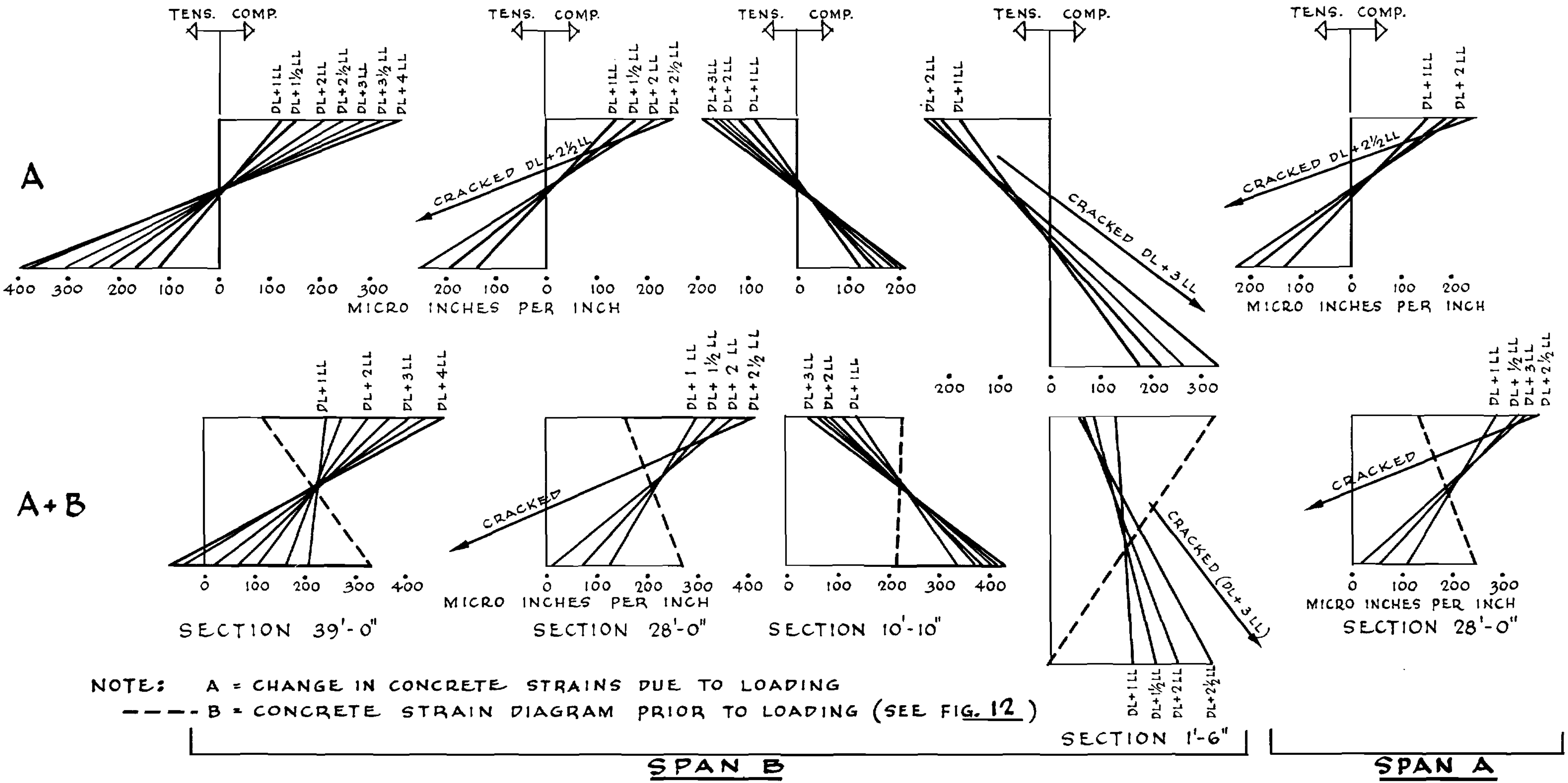
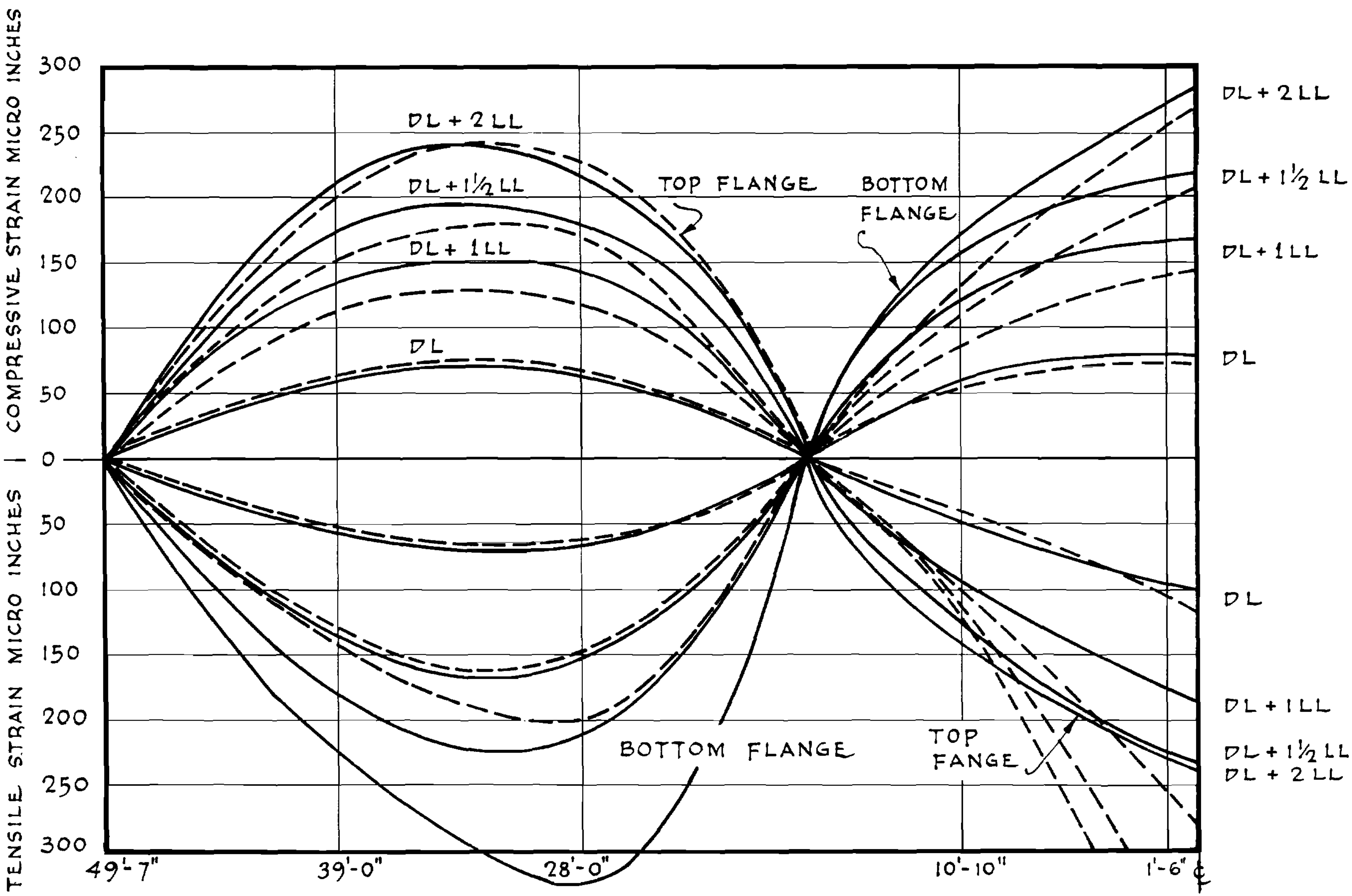


FIGURE 22

CONCRETE STRAINS DUE TO SYMMETRICAL LOADING

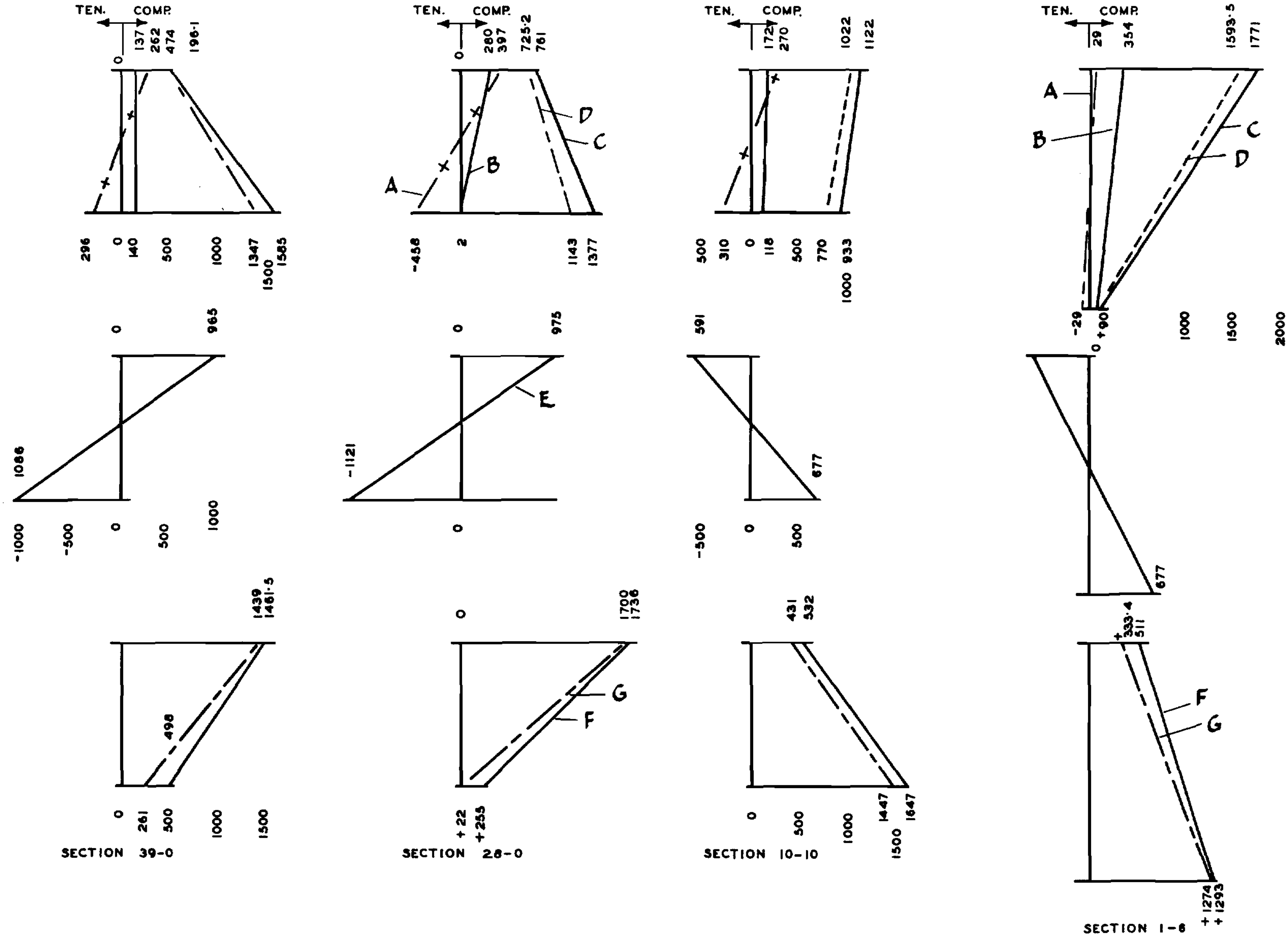


——— FROM BERRY GAUGE

– – – FROM ELECTRICAL RESISTANCE STRAIN GAUGES.

NOTE: ELECTRICAL RESISTANCE STRAIN GAUGE READINGS INVALID BEYOND DL + 1½ LL TENSION

FIGURE 23 CONCRETE STRAINS IN SPAN "B"



LEGEND

A = DEAD WEIGHT of 50 FT. BEAM

B = A + 100 % PRESTRESS of 50' WIRES

C = A + B + 100 % PRESTRESS of 100' WIRES

D = A + B + 85 % PRESTRESS of 100' WIRES

E = STRESS DIAGRAM FOR DL + 1LL

F = C + E

G = D + E

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FIGURE 24 DESIGNER'S THEORETICAL STRESSES

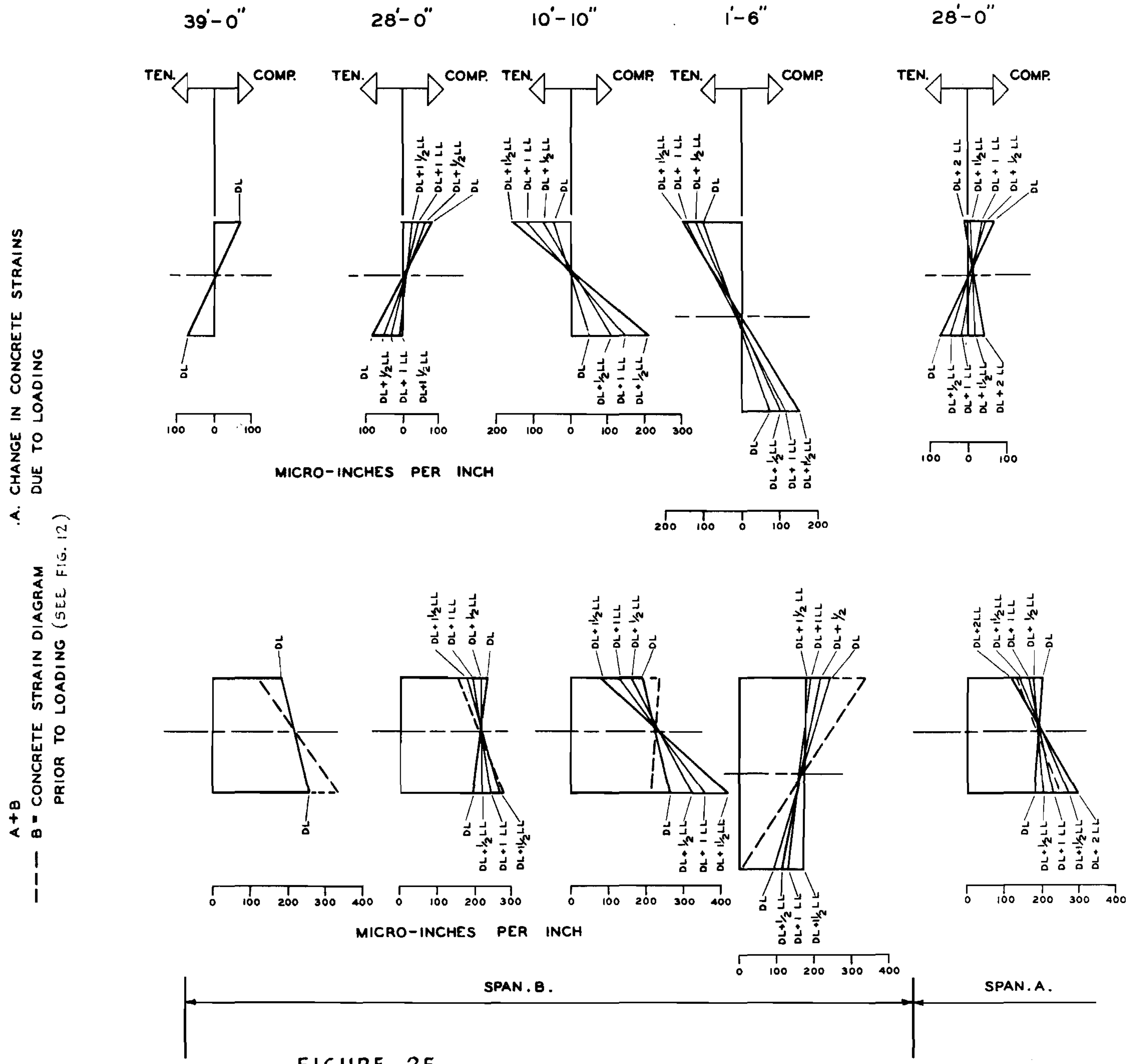

FIGURE 25

CONCRETE STRAINS DUE TO ASYMMETRICAL LOADING

CONCRETE STRAINS IN SPAN LOADED WITH CONSTANT LOAD OF DEAD LOAD OF ROOF STRUCTURE.

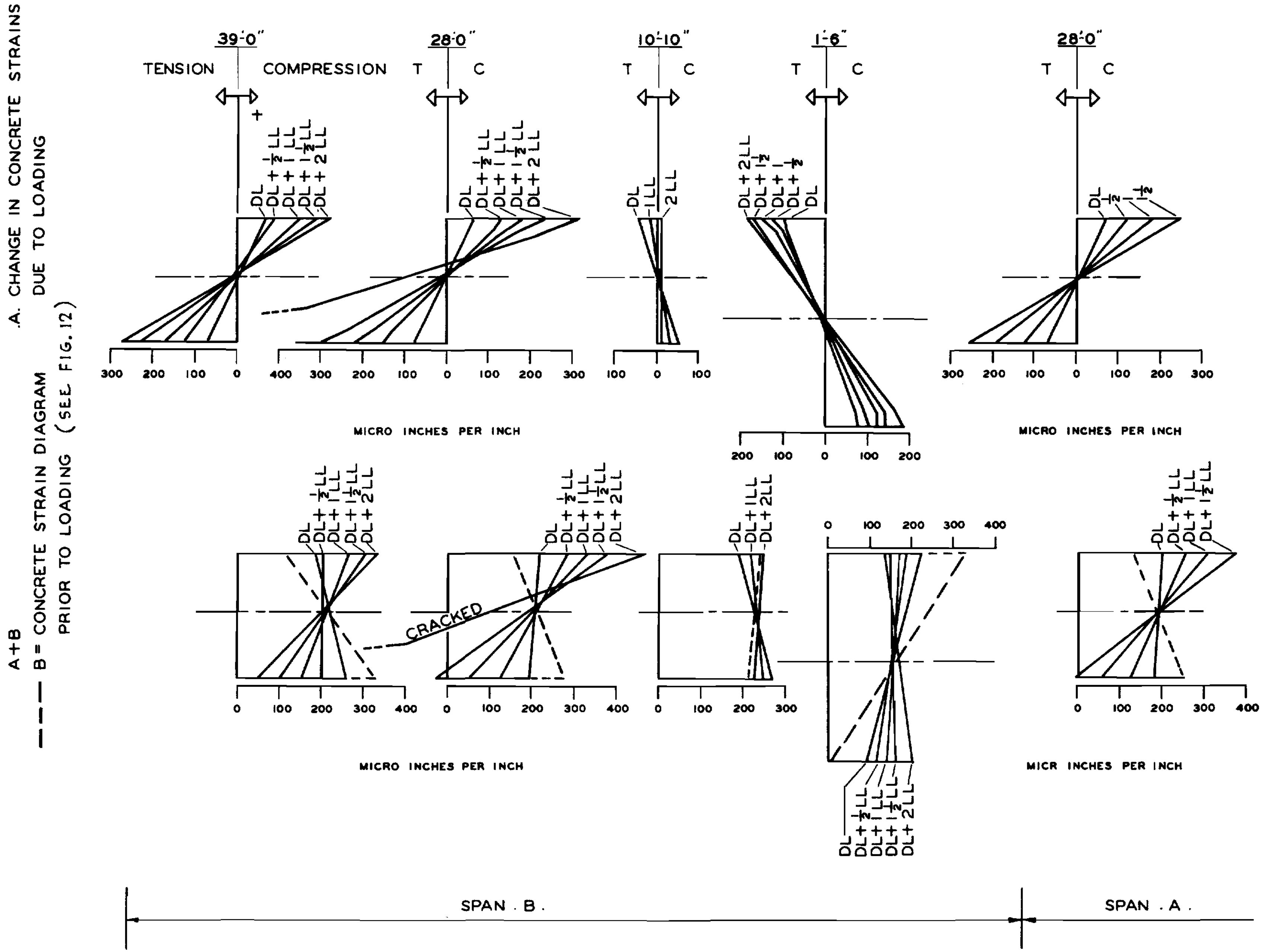

FIGURE 26

CONCRETE STRAINS DUE TO ASYMMETRICAL LOADING

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CONCRETE STRAINS IN SPAN LOADED WITH DEAD LOAD PLUS INCREMENTS OF LIVE LOAD.

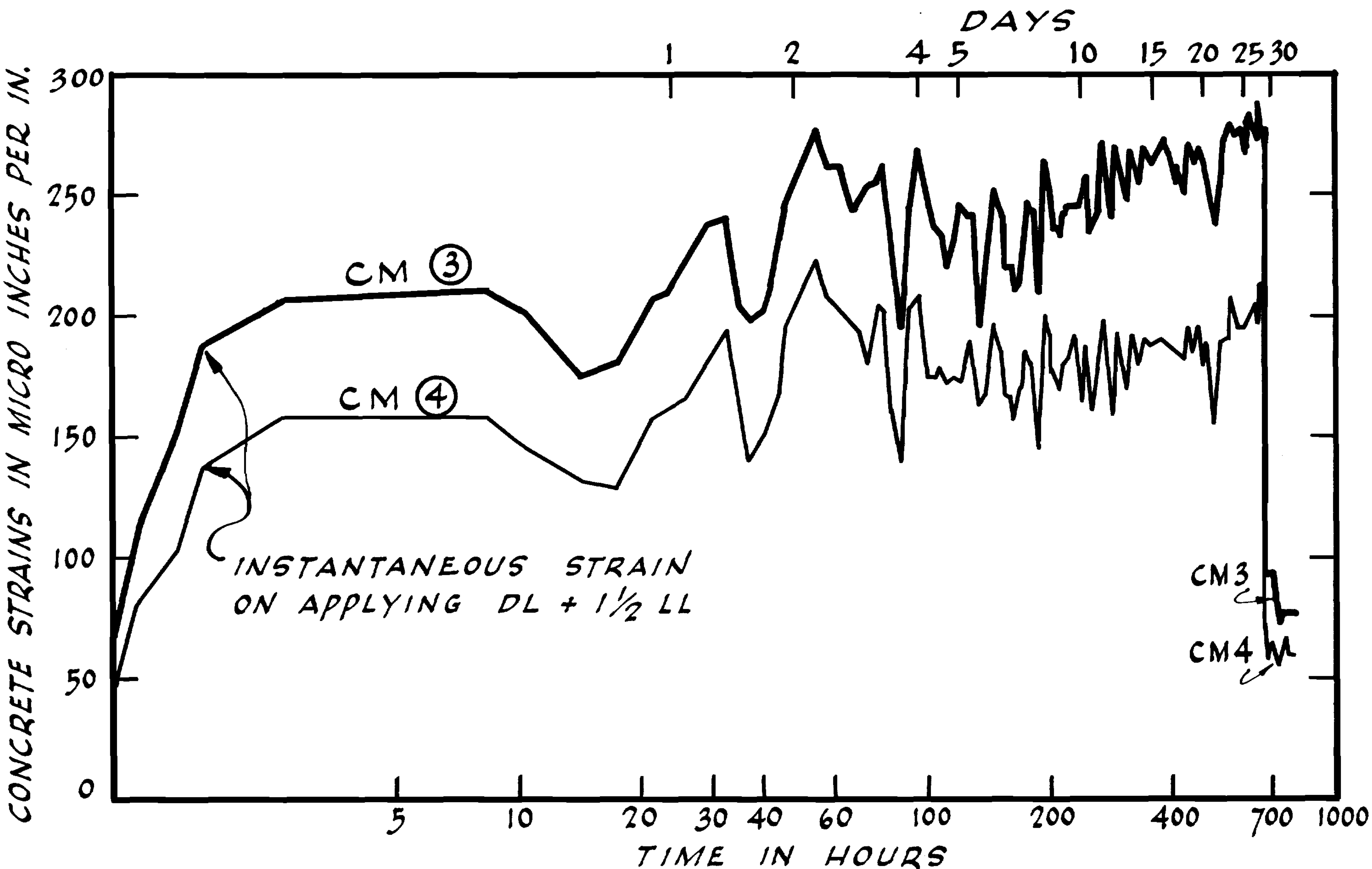

FIGURE 27

HISTORY of CONCRETE COMPRESSIVE STRAINS AT POSITIONS CM3 & CM4 DURING 28 DAY TEST

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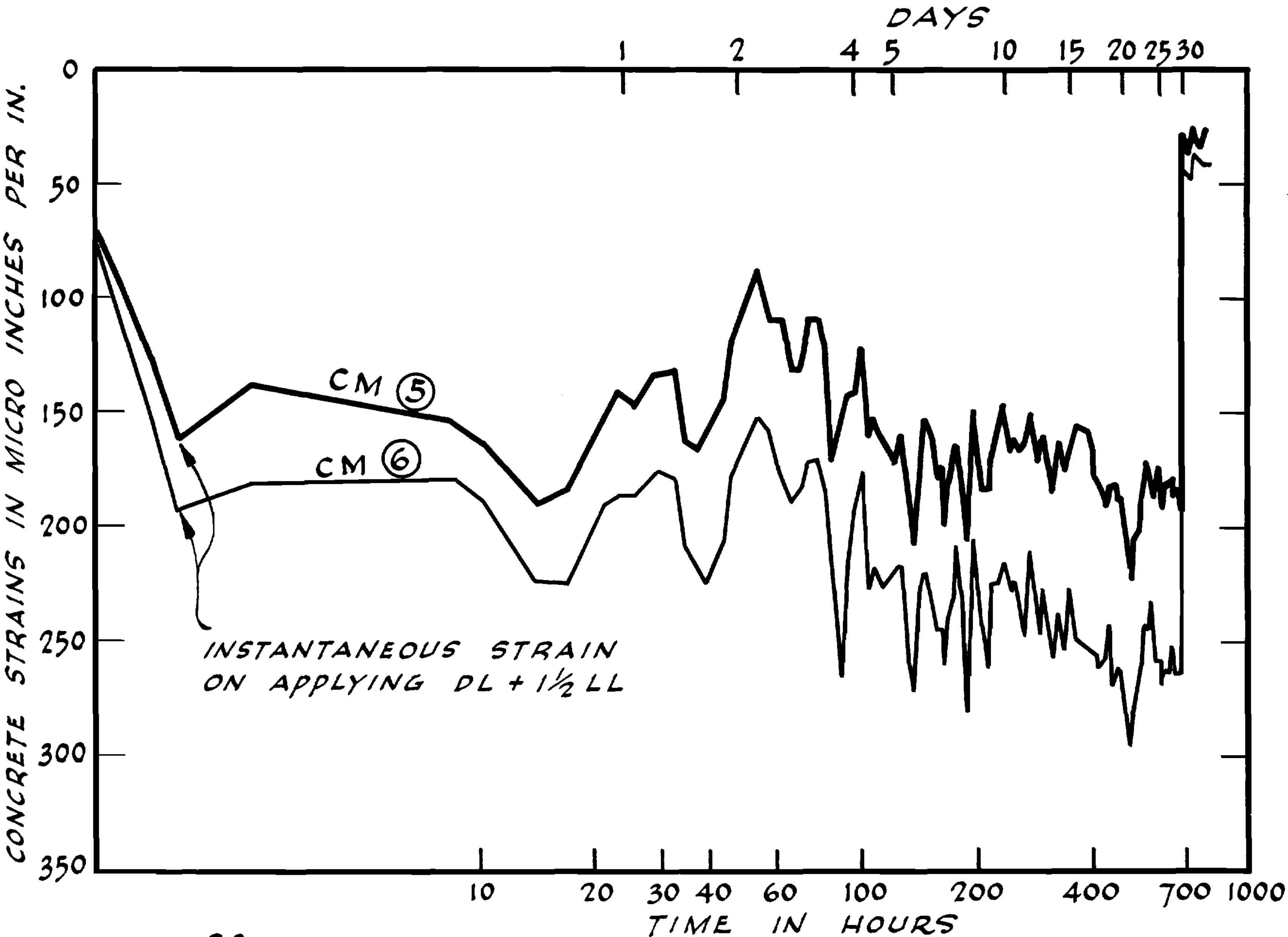


FIGURE 28

HISTORY of CONCRETE TENSILE STRAINS AT POSITIONS CM 5 & CM 6 DURING 28 DAY TEST

DBR RPT. 79

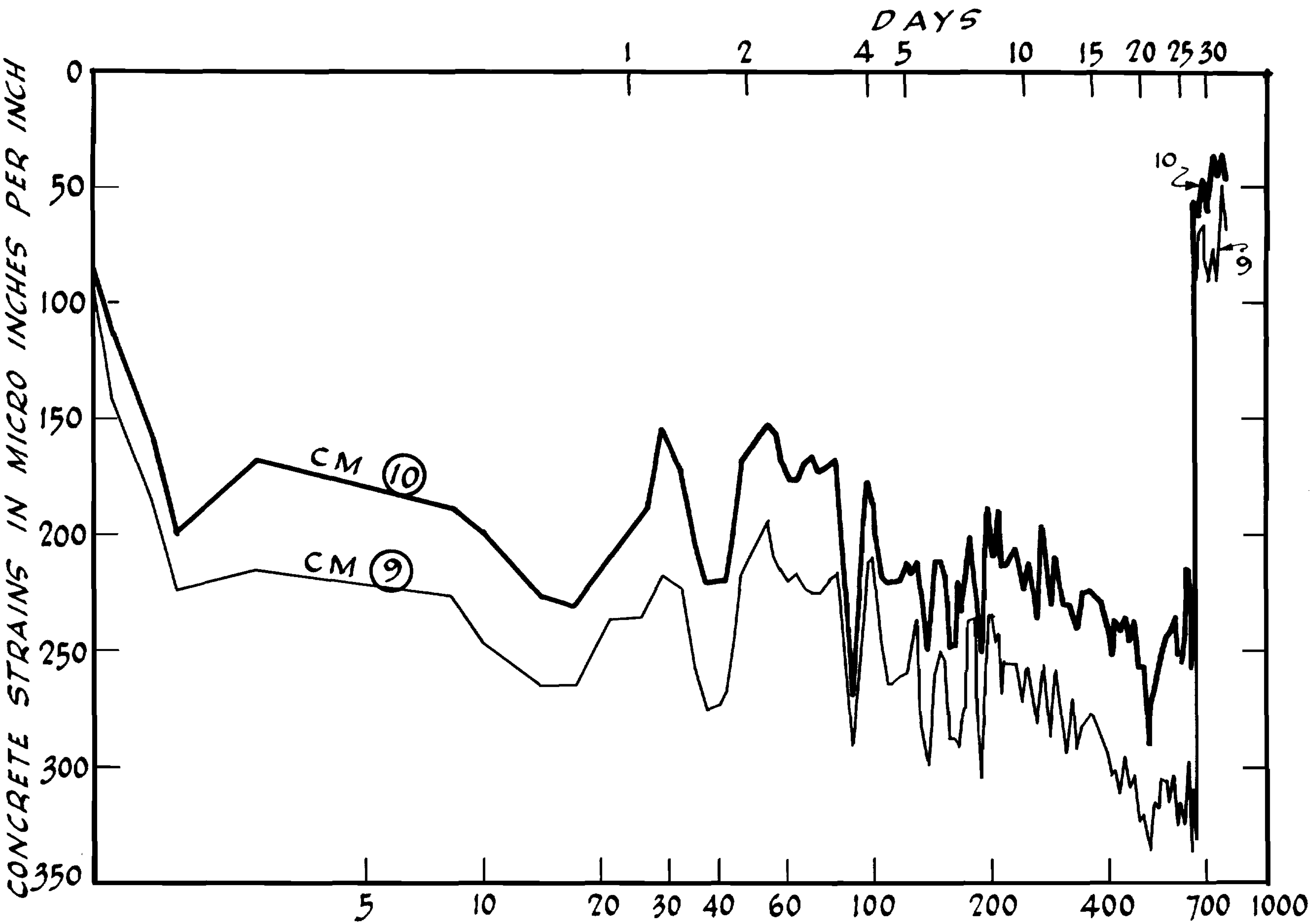


FIGURE 29

HISTORY of CONCRETE TENSILE STRAINS AT POSITIONS CM9 & CM10
DURING 28 DAY TEST

DBR RPT. 79

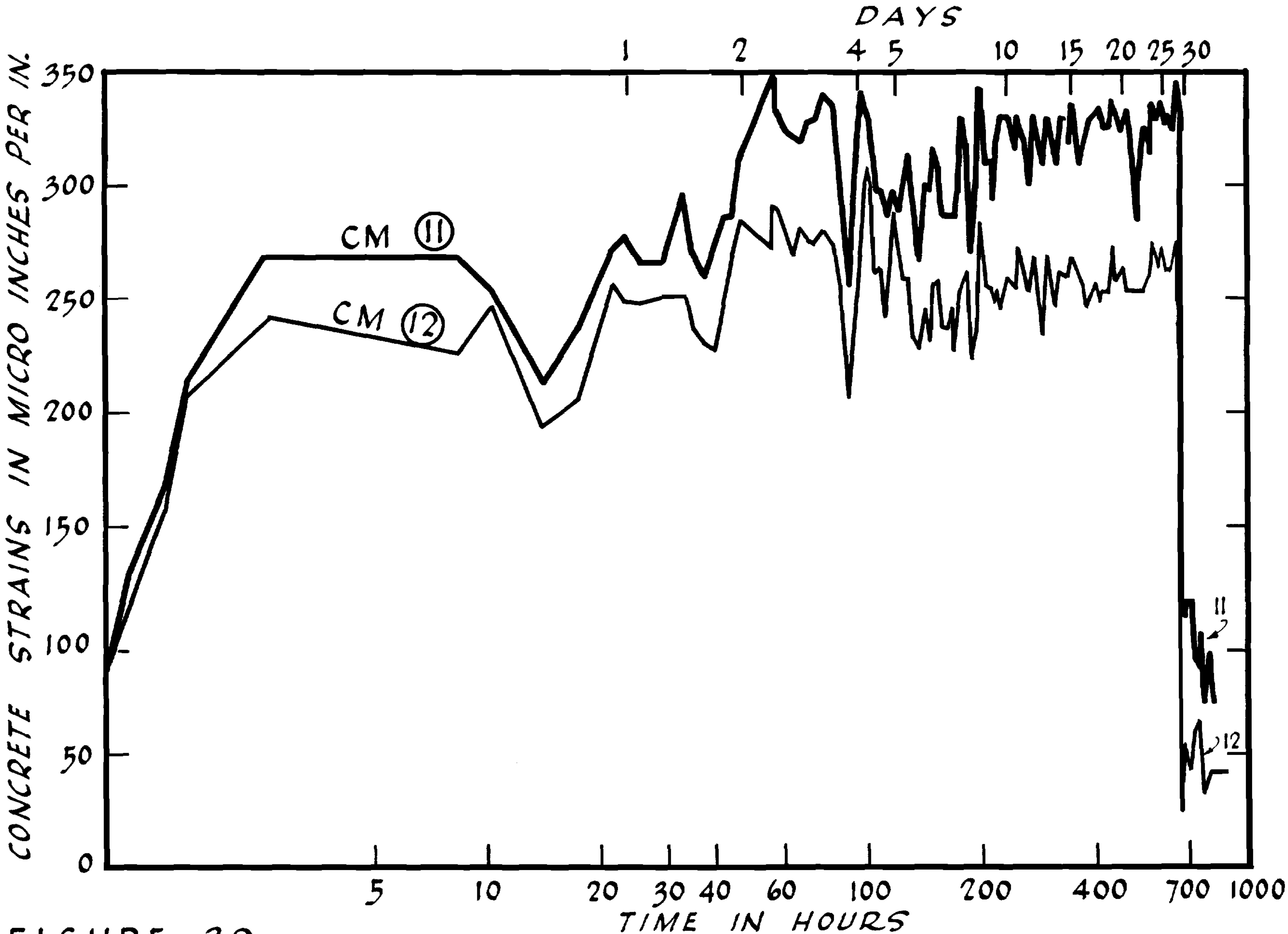
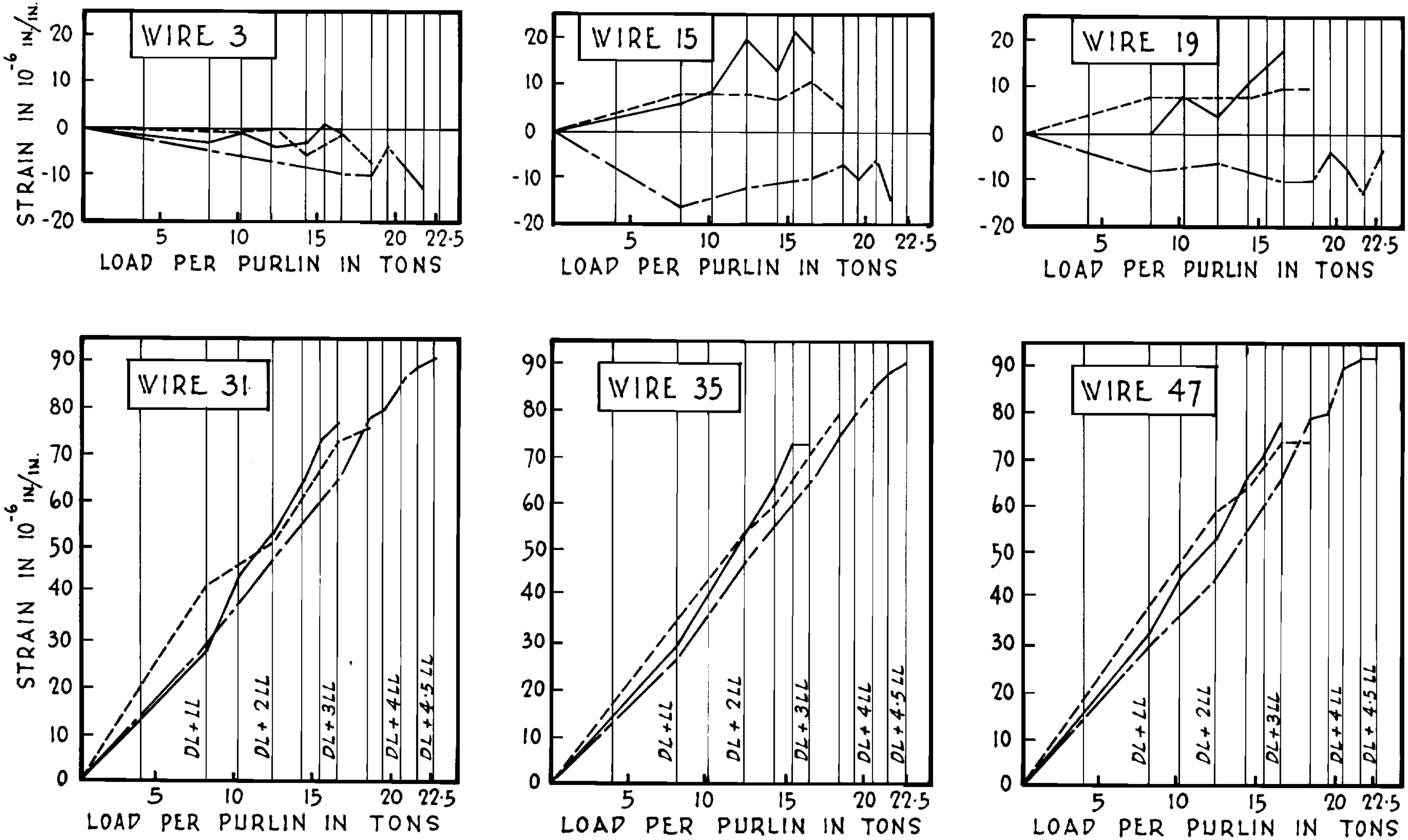


FIGURE 30

HISTORY of CONCRETE COMPRESSIVE STRAINS AT POSITIONS CM 11 & CM 12 DURING THE 28 DAY TEST

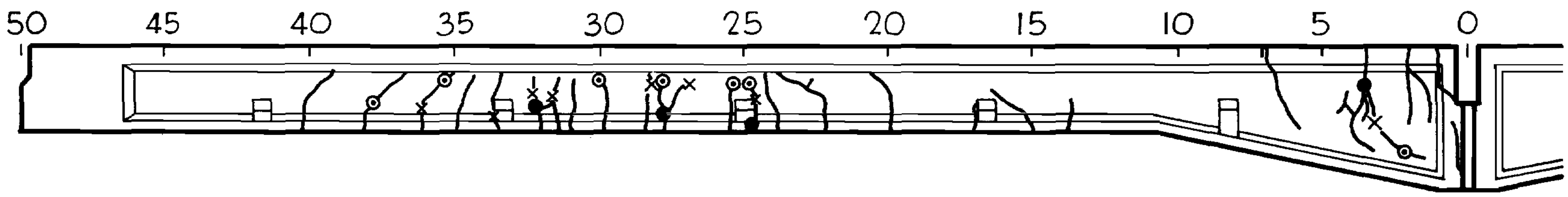


HISTORY of CHANGES IN STRAIN of 6 WIRES DURING LAST THREE LOAD SERIES

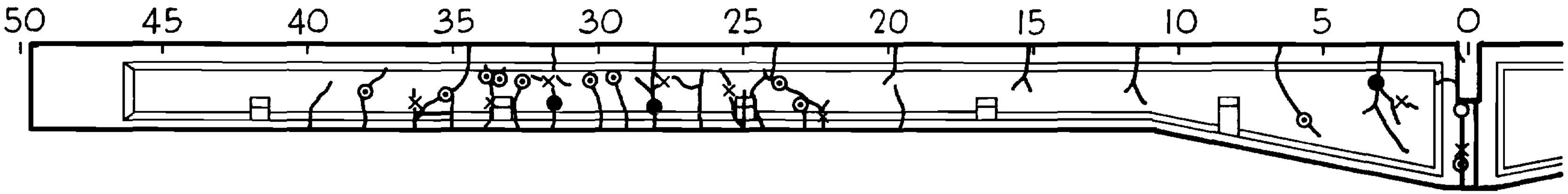
FIGURE 31

LEGEND:

————	LOAD	SERIES	12
– – – – –	"	"	13
—·—·—	"	"	14



SPAN "A"



SPAN "B"

LEGEND

END OF CRACKS AT VARIOUS LOADS

	LOAD No	LOAD
○	11·6	DL+ 2½ LL
●	12·8	DL+ 3 LL
×	13·9	DL+3½ LL
⊙	14·12	DL+4¼ LL

FIGURE 32

FINAL CRACKING PATTERN OF BEAM

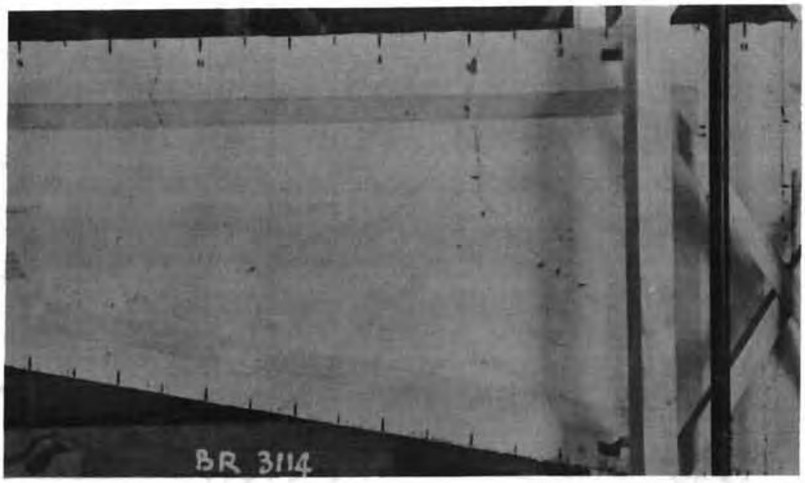
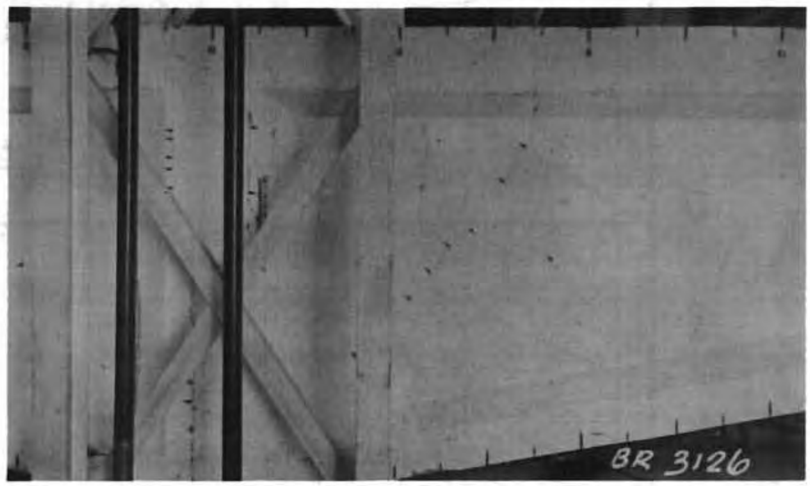


Fig. 33 Cracking at Haunch End of Span B.

Fig. 34 Cracking at Haunch End of Span A.



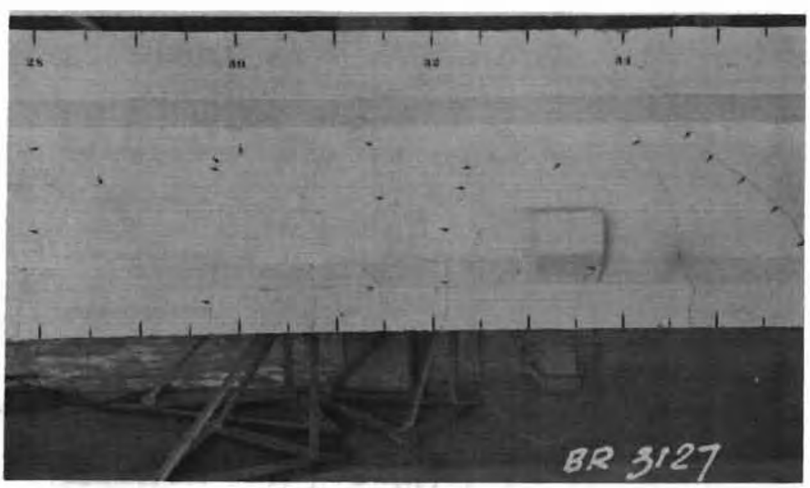


Fig. 35 Cracks in Span A.

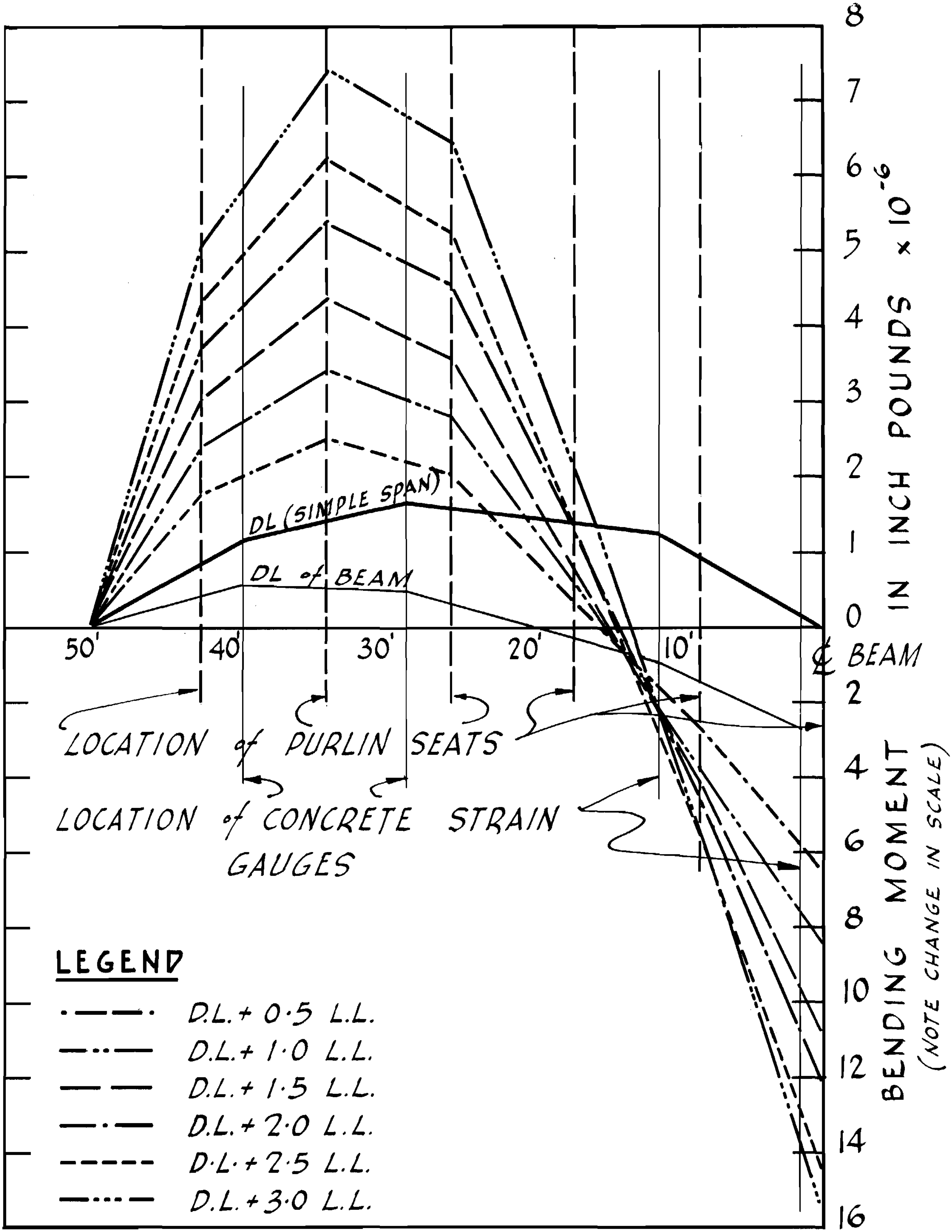


FIGURE 36

BENDING MOMENTS FOR VARIOUS LOADS

Fig. 37 Test Set Up Used for Testing Purlin Seats.

Fig. 38 Cracking at Purlin Seat at DL + 13 LL.

Fig. 39 Reinforced Concrete Beams Used in Similar Warehouse.