Tests of a 100-ft Continuous Prestressed Concrete Beam

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BY

L. J. MARCON AND W. G. PLEWES

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PREFACE

Prestressed beams used in 1953 as the main roof support for 200-by 500-warehouses in Cobourg, Ontario, represented an early use of prestressed concrete in Canada. It was because of this that the contract included the requirement that a full-scale load test to destruction of one of the 100 ft long prestressed beams, continuous over three supports, be made. The test was carried out by the Division of Building Research at the request of Central Mortgage and Housing Corporation and Defence Construction(1951) Limited, acting for the owners, the Department of National Defence.

Although the load test of the prestressed beam described in this report was carried out in 1953, the report of the results has not been available for public distribution. The test results may have lost some of their news value because of the increased use and more widespread knowledge in the art of prestressing today, but the tests have historical and engineering value. Because of this the results should be made available for general distribution. Furthermore, it is thought that continuous prestressed beams over three supports of 100-ft length have seldom been tested anywhere. The one tested showed that it could carry more than $5\frac{1}{2}$ times the design load before failure. The study included measurements of load-carrying capacity, deflection and strains in concrete and steel under a wide range of load conditions.

The Division of Building Research is grateful to the many who assisted in the execution of this project. As the work progressed, 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 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 1965

Robert F. Legget,
Director

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MISE À L'ESSAI D'UNE POUTRE CONTINUE DE 100 PIEDS EN BÉTON PRÉCONTRAINT

par

L. J. Marcon et W. G. Plewes

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On décrit dans ce rapport la mise à l'essai d'une poutre continue de 100 pieds (deux travées de 50 pieds) en forme d'I et en béton précontraint au moyen de 56 fils d'acier très souples ancrés selon le système Magnel-Blaton. Plus de cent poutres précontraintes de ce type ont constitué les poutres principales pour la toiture de quatre grands dépôts de l'intendance militaire à Cobourg dans l'Ontario.

La charge d'essai a été appliquée aux points de panne au moyen de onze vérins hydrauliques de 50 tonnes fonctionnant contre un cadre de réaction de pont Bailey. On donne le détail des plans et de la construction de la poutre, la méthode d'essai, les instruments employés, les observations enregistrées et on interprète les résultats obtenus. Les charges appliquées à la poutre étaient: des chargements symétriques et asymétriques sur les deux travées; l'essai du Code national du bâtiment (1941), un essai de charge à long terme (28 jours) et une charge finale de chargement statique plus une surcharge de 5.5.

On a établi que la poutre aurait failli sous environ six fois la charge calculée. La capacité de support de charge des crochets de panne de la poutre a également été déterminée au moyen d'essais.

TESTS OF A 100-FT CONTINUOUS PRESTRESSED CONCRETE BEAM

by

L. J. Marcon* and W. G. Plewes

SUMMARY

This report describes the testing of a 100-ft, continuous (two 50-ft spans), I-shaped concrete beam, prestressed with 56 high-tensile steel wires anchored by the Magnel-Blaton system. One hundred and fifty-two 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 (1941) 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.

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TESTS OF A 100-FT CONTINUOUS PRESTRESSED CONCRETE BEAM

by L.J. Marcon and W.G. Plewes

History of the Project

In 1953 a 500-ft-long, 200-ft-wide warehouse was built for Defence Construction (1951) Limited at Cobourg, Ontario. The roof system involved 100 two-span continuous beams which had originally been designed to be of ordinary reinforced concrete construction. At the suggestion of the contractor the design was changed to prestressed concrete and since, up to that time, prestressed concrete had been little used in Canada, this was something of an innovation. Also this conversion to some extent dictated the size and shape of the beams. For these reasons the owners and their inspection representatives, Central Mortgage and Housing Corporation, asked the Division to conduct a full-scale loading test on a typical girder. This the Division undertook as a co-operative project with the Ontario Hydro. (The Research Division of Ontario Hydro had previous experience in testing prestressed girders and were familiar with the problems involved in rigging, load application instrumentation, and test procedure. This experience was of considerable value.)

Description of the Warehouse

The warehouse roof construction consisted of precast, prestressed, concrete girders at 25-ft centres spanning the width of the building, and precast, reinforced concrete purlins running in the longitudinal direction. The girders were supported on cast-in-place, reinforced concrete columns at 50-ft centres, thus forming four 50-ft bays across the building width. Across the first and second, and across the third and fourth bays two 50-ft units were prestressed together to form two-span continuous beams 100-ft long.

The girders had an I-shaped cross-section with end blocks at the freely supported ends (Figure 1). Their depth was 3 ft, except for a 10-ft length over the centre support where haunching increased the depth to 5 ft. The top and bottom flanges were both 18 in. wide, the web 8 in. thick. Except on column lines, purlin brackets at 8-ft 4-in. centres were provided to support 25-ft reinforced concrete purlins, which in turn carried the light-weight concrete roof slabs.

On the column lines, rectangular purlins 9 in. by 32 in. framed into the continuous prestressed concrete beams at the column heads. They were seated in concrete pockets formed by block-outs at the ends of the prestressed girders, and to provide longitudinal rigidity negative reinforcement projecting from the ends of the purlins on either side of the girder was welded together and the pocket filled with cast-in-place mortar. All other purlins are simply supported on the beam brackets and were of a T-shaped cross-section with a depth of 23 in., a width at the top flange of 9 in., and a web thickness of 3 in. The main section properties are listed in Table I.

Construction and Prestressing of the Main Beams

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

Before erection, two 24-wire cables 100 ft long were inserted in the longitudinal ducts in one of the 50-ft beams. The 50-ft lengths 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-ft units had been lifted to the top of the columns, with their haunched ends meeting over one column, the 4-in. gap between the haunched ends was partially grouted in with a high-strength, quick-setting expanding grout, leaving a U-shaped gap to form a duct for the prestressing wires. This duct was continuous with similar U-shaped slots in the ends of the 50-ft beam sections.

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 pulled through the appropriate duct. The beam was then ready to be tensioned. When the concrete had reached its specified 5000-psi strength, the cables were 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 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-ft cable was raised at the centre support. Four 1-in. diameter pins were placed through the web of the beam to hold the cable in this deflected position (Figure 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 containing about 1 cu ft of cement, 1 cu ft of sand and 5 gal of water was used; but it was unsatisfactory because it did not always fill the full length of the ducts by gravity, probably as a result of sand particles in the ducts. The remaining spaces were later filled by pressure grouting with a liquid grout without sand.

In subsequent beams a water-cement grout proved satisfactory. The mixture was 5 bags of cement to 27 gal of water. For the complete grouting of the 100-ft 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.

Design and Load Requirements

The beams were designed to carry a roof life load of 40 lb/sq ft. 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-ft main girders weigh approximately 26 tons each (Table II).

TESTING PROCEDURE

General

In view of the fact that the test beam was made under normal field conditions and not under conditions that could be called "controlled" from a research point of view, it was not considered that the project warranted the extensive instrumentation required for a complete strain investigation. A limited number of strain measurements were, however, made

The arrangement used to test the beam consisted of hydraulic jacks and a reaction beam made up of Bailey bridge units (Figure 2). The method of applying load and anchoring the test beam and the reaction beam is shown in Figures 2(b), 3 and 4. The centre support was fixed, whereas both end supports were on steel rollers.

Loads

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

Full-scale reading on the oil pressure gauges on the pumps was 10, 000 psi, and difficulty was experienced in applying small loads as low as 4, 000 lb per jack ($\frac{1}{2}$ DL) because of the very low pressures to be used on the large jacks. Very good agreement between the jack loads and the reactions was, however, obtained at loads equal to and greater than DL + LL.

Deflections

Deflections were recorded at each purlin point and at both ends, thus giving a total of 13 points (Figure 6); three methods of measuring deflection were used.

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

Very fine wires attached to the under side of the beam and passed over a system of pulleys to a system of weights and a marking board at a central location provided a means for measuring the larger deflections under higher loads. This method has important advantages in that any progressive deflection under load can be detected immediately and remote measurement is possible.

Further deflection readings to measure any settlement were made with a precise optical levelling instrument at the ends and centre support of the beam. To check any possible lateral rotation, dial gauges were placed at the outer edges of the flanges at the ends and centre of the girder. In addition, two spirit levels were placed at the mid-span locations to indicate tipping.

Steel Strains

Strains in 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 on which gauges were located are shown in Figure 7.

Concrete Strains

During casting, more concrete test cylinders were taken from span B than from span A, so that 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, as a check on the strain readings, by an 8-in. mechanical extensometer straddling the gauge points (Metzger gauge). The positions of the gauges are shown in Figure 6.

PROPERTIES OF THE TEST SPECIMEN

General

The two 50-ft units that made up the 100-ft continuous test beam were cast and tensioned on the following dates:

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

Concrete Data

The concrete mix contained four bags 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 in. Specifications required a minimum compressive strength of 5,000 psi at seven days. At the time of test the cylinder strengths were:

	Age of Cylinders at Start of Beam Tests	Compressive Strength
Span A	106	6,700 psi
Span B	134	7,200 psi

The concrete in span B had a slightly higher compressive strength than that in span A, but the average strength for both spans was 7,000 psi.

Modulus of Elasticity of Beam Concrete

The conversion of the concrete strain data to equivalent stresses requires a knowledge of 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. A series of compression cylinders and concrete core samples taken from the beam were tested under different loads. Three concrete cylinders tested at the conclusion of the beam test gave an average E_c of 5.4×10^6 psi. Concrete core samples (2.8 in. in diameter, 5.6 in. high, age 11 months) taken from a section of span B gave an average E_c of 5.0×10^6 psi. From this evidence 5.2×10^6 psi was taken as a representative value.

Steel Data

The specifications for the prestressing wire were as follows:

(i) minimum yield strength (0.2 per cent offset) 160,000 psi,

(ii) minimum ultimate tensile strength 215,000 psi, and,

(iii) working stresses not to exceed 128,000 psi.

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

PRESTRESSING OF THE BEAM

General

No strain and deflection measurements could be made during the initial partial prestressing of the 50-ft 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-ft wires required an elongation of 3 in. and that the average slip occurring during the wedging operation was 1/8 in.

The beam was ready for complete tensioning of the 100-ft wires on 11 May. Upon tensioning the wires, however, it was found that abnormally high pressures were necessary to obtain the required 6-in. elongation. The condition became so serious that it was decided to replace completely the 100-ft 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 152 beams tensioned that showed such an irregularity. A new pair of cables was inserted and new electrical resistance gauges had to be installed.

On 25 May the new wires were tensioned without difficulty; Figure 7 shows the sequence in which this was done. The same day the lower cable was raised by a jack and sling arrangement and the location pins were inserted.

Deflections During Prestressing

The upward deflections of the beam during tensioning are shown in Figure 8; the maximum recorded is 0.15 in.

During the tensioning operation span A always showed a greater upward deflection than span B. As the stressing was done from the free end of span A, this condition was probably caused by the loss of prestress due to friction along the cable duct that resulted in a smaller prestressing force in span B. The difference in deflection could also be attributed to difference in age, because span B was the older span. During prestressing there was very little movement at the ends and centre support, which lifted about 0.004 in. The camber behaviour with time could only be followed for one day, because erection of Bailey test structure made the removal of the dial gauges imperative. Figure 9 shows the time deflection curve for two similar points on the beams.

Steel Strains During Tensioning of 100-ft Tendons

During tensioning of the 100-ft wire tendons the difference in their elongations before and after wedging was measured with a steel rule. The results are shown in Table III. The slip varied from 1/16 to 1/2 in., with an average of 1/8 in. Similarly, strains in the wires carrying electric resistance gauges were recorded before and after wedging and the results are tabulated in Table IV.

In addition, readings were taken on all gauged wires after each sandwich plate or eight wires had been completely tensioned. The adjustment of steel tension among the wires can be seen in Table V.

Strains in the wires during the first $4\frac{1}{2}$ days are also shown in Figure 10, which indicates the effects of lifting of cables, as well. Eight of the eleven gauged wires were tensioned higher than the 128,000 psi stipulated in the specifications. The maximum stress recorded was 156,000 psi, or 121.5 per cent greater than 128,000 psi. This maximum stress is 77 per cent of the average measured yield strength of 203,000 psi, and 68 per cent of the average ultimate tensile strength of 229,400 psi. The average steel strain after final prestressing was 5,000 micro-in. per in., or 139,000 psi. This average measured stress of 139,000 psi is 68.5 per cent of the yield point stress and 60.7 per cent of the ultimate tensile stress.

Unfortunately, the grouting of the cable ducts made the gauges unstable for a period of two weeks and it was not possible to follow directly the loss of stress in the steel following prestressing. After a two-week period the gauges again stabilized and new zeros were taken before the load test program.

Concrete Strains During Tensioning of the 100-ft Wires

Concrete strain readings were taken before, during, and after the tensioning of the 100-ft wires. The concrete strain history of the beam is shown in graphical form in Figure 11. Because of temporary malfunction of the electric resistance gauge instrumentation at this stage only the mechanical extensometer readings were used.

The strain patterns for the same sections of spans A and B (28-ft mark) are very similar and serve as a good check on the readings.

The bottom row of strain diagrams in Figure 11 also shows theoretical strains obtained using an E_c of 5.2×10^6 psi. The largest variation occurred at the 10-ft 10-in. section.

The dotted line, C, in Figure 11 is the total of the theoretical design figures for the dead weight of the beam and 100 per cent prestress of the eight 50-ft wires, plus the measured strains due to prestressing of the forty-eight 100-ft wires. This last total will be used for the discussion of the final strain conditions in this report.

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 was 117 micro-in. per in. This increase in strain from creep and shrinkage represents a loss of 2.3 per cent of the average steel wire strain of 5,000 micro-in. per in. at the time of final prestressing.

The total average increase in concrete strains up to the final day of testing was 166 micro-in. per in., 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.

TEST PROGRAM AND LOADING SCHEDULE

In order to study the behaviour of the beam under various loading conditions the loading program in Table VI was carried out. In this report dead load (DL) always represents the weight of the roof structure, excluding the weight of the beam itself - that is, the weight of the purlins, roof slabs and roofing materials. Instrument readings were taken before and after the application of all load increments.

National Building Code

As there were 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 specified that a load equal to dead load plus $1\frac{1}{2}$ live load be left on the beam for 24 hours and that the beam have at least a 75 per cent deflection recovery in the 24-hour period following removal of the load. Deflection, and steel and concrete strain readings were taken at regular intervals during and after load was applied.

Asymmetric Loadings

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

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 for several days after the removal of load.

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 it 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 upward springing of the beam.

After consideration of all factors it was decided not to rearrange the entire test set-up 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.

Although the maximum test load applied was DL + 5.5 LL, the slope of the load deflection curve (Figure 12) at DL + 5.25 LL indicates that the beam was very close to 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-ft 6-in. 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, the probable ultimate failure load from DL + 5.5 LL to DL + 6.0 LL.

DEFLECTION BEHAVIOUR DURING TESTING

Symmetrical Loadings

The complete deflection data obtained from the wire and pulley system are plotted in Figure 13. This figure gives the deflection pattern up to the last measured readings. Readings were not available for DL + 5.5 LL because the jacks became displaced just as this load was reached. For a more accurate picture of deflection see Figure 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.

Up to DL + 1.5 LL span A showed the larger deflection. For DL + 2 LL deflections for the two spans were almost equal and for loads larger than DL + 2 LL span B showed the larger deflection. This behaviour was also 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 the cracking pattern indicated that it would be span B.

Figure 12 is the load deflection curve for the beam at positions 9 and 10. 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. Cracking occurs at or about the junction of the two portions of the curve. 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 in. at gauge 10 on span B. (i.e. about 1/180)

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

The end of span A had a downward movement of about 0.06 in. 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 in. and had to be corrected by adjusting the nuts on the tie rods. With increased loading the upward deflection remained at about 0.30 in. The end of span B had a constant upward deflection of about 0.03 in. up to DL + 2 LL, increased to 0.09 in. for loads up to DL + 4 LL, and at a load of DL + $4\frac{1}{2}$ LL increased to 0.30 in. and remained there until the 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 cracked. This is clearly shown in Figure 15. 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 in. for loads up to DL + $2\frac{1}{2}$ LL and increases to 0.10 in. at DL + $3\frac{1}{2}$ LL.

Comparison of Actual and Theoretical Deflections

In Figure 16 the calculated and measured deflections for a load of DL + 1 LL have been plotted. The values were obtained from the design calculations for 100 per cent and 85 per cent prestress.

The maximum calculated deflection at the 32-ft mark (0.155 in.) at 100 per cent prestress was 75 per cent of the actual measured deflection (0.206 in.) (average of spans A and B) and 84 per cent for the calculated deflection at 85 per cent prestress (0.173 in.). Seven ft from the centre support the calculated deflection at 100 per cent prestress (0.005 in.) was 20 per cent of the actual measured deflection (0.025 in.) and 32 per cent of the calculated deflections at 85 per cent prestress (0.008 in.). These differences appear to be due to an overestimate of the stiffness of the haunched section.

Deflections During National Building Code Test

The deflections of two identical points on spans A and B are plotted in Figure 17 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 and 98.5 per cent. The beam, therefore, passed the required minimum deflection recovery of 75 per cent.

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 Figures 18 and 19.

The irregularities in the deflection curve are the result of 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 in. and increased to 0.384 in. or 49 per cent in 28 days. The immediate deflection on removing load was 0.128 in. or a recovery of 66.6 per cent. During the 5-day recovery period the deflection decreased to 0.095 in. giving a total recovery of 75.2 per cent. Similar deflection patterns were obtained for all other deflection gauges. The time and sequence of rise and fall in the curve are the same as those of the concrete strains during the 28-day test (Figures 20 - 23).

Deflection of Beam During Asymmetrical Loadings

The deflections of the beam under asymmetrical loadings are plotted in Figure 24. 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 as a result of uplift of the span.

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

CONCRETE AND STEEL STRAIN BEHAVIOUR DURING TESTING

Symmetrical Loadings

The concrete strains recorded during the symmetrical loadings are plotted in Figure 25. These values represent averages of all readings obtained from the particular points.

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

The strain diagrams at the 28-ft section of span B (4 ft 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-in./in. At DL + $2\frac{1}{2}$ LL the beam had cracked at this location and gave an extremely large strain reading as shown. Similar behaviour was recorded at the 28-ft section of span A.

At the 1-ft 6-in. section the cracking occurred at DL + 3.0 LL, a loading condition at which (according to the strain diagram, Figure 11) 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-ft wires. Theory assumed uniform stress across the 1-ft 6-in. section, which was only 1 ft 4 in. from the end anchorages of the 50-ft wires. To make the strain diagram agree with the tensile strain in the top fibre that 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-ft sections. A comparison of strains measured by extensometer and electrical resistance strain gauges is made in Figure 26.

Theoretical Stresses

The theoretical stresses of the beam for various load conditions are plotted in Figure 23. 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 with measured strains. As 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 (Figures 25 and 27). 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 x 10^6 psi, with an average E_c of 6.3 x 10^6 psi. This degree of variation makes difficult an accurate comparison of stresses and strains.

Concrete Strains Due to Asymmetrical Loadings

In Figures 28 and 29 the concrete strains due to asymmetrical loadings are plotted. In Figure 28 the concrete strains of span B loaded with constant DL are shown for DL plus various increments of LL on span A. In Figure 29 the concrete strains from span B loaded with DL plus increments of LL (with DL only on span A) 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-ft 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-and 28-ft sections. At the 28-ft section the asymmetrical loading up to DL + 2.0 LL produced a change in compressive strain of 320 micro-in./in. in the top fibre, whereas the change due to the symmetrical loading of DL + 2.0 LL was 215 micro-in./in. At the 1-ft 6-in. section the change in concrete tensile strain due to DL + 2.0 LL was 235 and 190 micro-in./in., 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-ft 10-in. section, which under symmetrical loading was the location of the least concrete strain changes. The concrete strain changes, A, at the 1-ft 6-in. and 10-ft 10-in. sections increased with loading on the opposite span, whereas the concrete strains at the 28- and 39-ft sections decreased. The concrete strain changes due to increasing load at the 39-ft 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.

Concrete Strains During National Building Code Test

The time-strain curves obtained for the National Building Code test follow a pattern similar to the time-deflection curve shown in Figure 17. 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.

Concrete Strains During the 28-day Test

For the 28-day test, only the mechanical extensometer readings were used in order to avoid possible zero drift with time in electrical resistance strain gauges.

Typical histories of the concrete strains during the 28-day test are shown in Figures 20 - 23. The figures present strains at the 28- and 1-ft 6-in. 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. Table VII shows the percentage changes in concrete strains during and after the 28-day test. The concrete stresses calculated from the strain diagrams in Figure 25 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 VII shows that strain gauges CM 3, 4, 11 and 12 represent the deflection behaviour more closely than those 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, due probably to small stresses present at the gauges.

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 were repeated after application of each load. The differences between these readings for six typical wires are plotted in Figure 30 for the last three load series.

Wires 3, 15 and 19 showed a very irregular strain pattern because of their position above 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 Figure 30.

The maximum recorded increase in steel strain (in wire 47) was 92×10^{-6} in./in. 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 the position of maximum moment.

CONCRETE CRACKING DURING TESTING

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 in. (when first noticed) to 21 in. at the end of the 28-day test. It reopened on applying DL + 1 LL. The first crack in the zone of positive moment occurred at DL + $2\frac{1}{2}$ LL at the 31-ft 6-in. mark on span B, and 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-ft marks. No cracking was noticed in span A.

At DL + 2 3/4 LL the lower flange of span A started to crack at 32 ft. The load of DL + 3 LL increased the length of existing cracks and produced an additional crack at the 28-ft 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 15 in. deep at the 3-ft 6-in. mark, while the crack in span B occurred at the 3-ft mark and was 20 in. deep. Further loadings increased the extent of existing cracks and produced the crack pattern shown in Figure 31. The largest crack observed before the jacks were displaced was at the 31-ft 6-in. mark on span B and would probably have been the location of ultimate failure. No visible cracks were noticed near either end of the beam. At the maximum applied load of DL + 5.5 LL the beam had almost reached its carrying capacity; 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-ft 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 Figures 32 and 33.

These cracks occurred at the joints formed between the precast beams and the grout used to fill the 4-in. gap at the centre. Their presence was one of the reasons for not continuing the test after the jacks kicked out. Further details of cracks are shown in Figure 34.

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 ft, followed by two others at 25 and 31 ft 6 in. The crack at 28 ft occurred between the two strain gauge points at that location.

No cracking occurred in span A during the asymmetrical loadings, but the above cracks reopened on applying a symmetrical load of DL + 2.5 LL.

END AND CENTRE REACTIONS

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. Greater variations also occurred 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 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 35).

Asymmetrical Loadings

Larger differences occurred between loads calculated from load cells and hydraulic jack pressures during the asymmetrical loadings than during the symmetrical loadings. The reason was probably the effect of the rotation of the beam at the centre support upon the load cells where there was no roller. Table VIII shows 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 are 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 that produced cracking in the span loaded with DL + 2 LL.

PURLIN BRACKET TESTING

Initially it was planned that the beam would be loaded 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 had failed before the beam itself, the damage might have been sufficient to prevent further loading of the beam. This would have been undesirable, since the main purpose of the test was to check the behaviour of the beam under load, not the behaviour of the purlins.

The purlin seats were loaded with the yoke arrangement shown in Figure 36. Two pairs of seats in span A were tested. The first pair, 16 ft 8 in. 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-ft 8-in. 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 Figure 37. Further purlin bracket testing was considered unnecessary.

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. Examination of the ducts at the ends of the cut sections revealed that all the wires were surrounded by grout. There was, however, a small narrow space between the grouted cable mass and the concrete edges of the ducts as a result of grout shrinkage. The maximum width of this space was about 1/16 in.

CONCLUSIONS

1. Although the loading system failed before the ultimate failure load of the beam was reached, the indications were that it would have failed completely at a load approaching DL + 6.0 LL.

2. The load factor at the maximum load reached in the test was

$$\frac{\text{Beam weight + DL + 5.5 LL}}{\text{Beam weight + DL + 1 LL}} = \frac{294.5}{114.6} = 2.57$$

3. The factor of safety at first crack over the centre support was 1.5 and in the span, 2.5.

ACKNOWLEDGEMENTS

Much assistance was received by the Division during this work, and it would be impossible to list all those who gave a helpful hand either in the work itself or in the form of suggestions during discussions. Special mention, however, must be made of the co-operation of the Hydro-Electric Power Commission of Ontario and, particularly, the staff of the Structural Research Department, without whose help the work could not have been carried out. The entire test project was made a co-operative venture between the Division of Building Research and the Structural Research Department of the Hydro-Electric Power Commission of Ontario.

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This paper is based largly on a private report prepared by the first author after the test for the Central Mortgage and Housing Corporation and Defence Construction Limited, and has now been adapted for publication in this form by the second author. The first author is a former, the second a present, member of the Building Structures Section of the Division of Building Research. The paper is published with the approval of the Director of the Division.

TABLE I

SECTION PROPERTIES OF BEAM AT CONCRETE STRAIN GAUGE LOCATIONS

Distance From Centreline 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 II

MAGNITUDE OF TEST LOADS

Dead Load (Roof Structure)	37.4 psf
Live Load (Snow Load)	40.0 psf

Description of Load	Lb/Sq Ft	Load Per Jack (lb)	Total Load (tons)
Dead load of roof Structure	37.4	7,780	42.7
Dead load + $\frac{1}{2}$ 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 III

LOSSES IN STEEL WIRE ELONGATIONS DUE TO WEDGING

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

Wire No.	Elongation Before Wedging	Elongation After Wedging	Loss	Wire No.	Elongation Before Wedging	Elongation 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 IV

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-in. per in.

TABLE V

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 hr	14 hr
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-in. per in.

TABLE VI

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 + $\frac{1}{2}$ LL	
		3.3	DL + 1 LL	
July 1	4	4.1	DL	
		4.2	DL + $\frac{1}{2}$ LL	
		4.3	DL + 1 LL	
		4.4	DL + $1\frac{1}{2}$ LL (National Building Code Requirement) Held for 24 hr	
			ASYMMETRIC LOADING	
			Span A	Span B
July 2	5	5.1	DL	DL
		5.2	DL + $\frac{1}{2}$ LL	DL
		5.3	DL + 1 LL	DL
July 3	6	6.1	DL	DL
		6.2	DL	DL + $\frac{1}{2}$ LL
		6.3	DL	DL + 1 LL
			SYMMETRICAL LOADING	
July 3	7	7.1	DL	
		7.2	DL + $\frac{1}{2}$ LL	
		7.3	DL + 1 LL	
		7.4	DL + $1\frac{1}{2}$ LL - Held for 28 days Recovery period 5 days.	
August 5	8	8.1	DL	
		8.2	DL + $\frac{1}{2}$ LL	
		8.3	DL + 1 LL	
		8.4	DL + $1\frac{1}{2}$ LL	
		8.5	DL + 2 LL	

TABLE VI (Cont'd)

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 + $\frac{1}{2}$ LL	DL
		9.3	DL + 1 LL	DL
		9.4	DL + $1\frac{1}{2}$ LL	DL
August 5	10	10.1	DL	DL
		10.2	DL	DL + $\frac{1}{2}$ LL
		10.3	DL	DL + 1 LL
		10.4	DL	DL + $1\frac{1}{2}$ LL
		10.5	DL	DL + 2 LL
SYMMETRICAL LOADING				
August 5	11	11.1	DL	
		11.2	DL + $\frac{1}{2}$ LL	
		11.3	DL + 1 LL	
		11.4	DL + $1\frac{1}{2}$ LL	
		11.5	DL + 2 LL	
		11.6	DL + $2\frac{1}{2}$ LL	
August 6	12	12.3	DL + 1 LL	
		12.4	DL + $1\frac{1}{2}$ LL	
		12.5	DL + 2 LL	
		12.6	DL + $2\frac{1}{2}$ LL	
		12.7	DL + 2 3/4 LL	
		12.8	DL + 3 LL	
August 6	13	13.3	DL + 1 LL	
		13.5	DL + 2 LL	
		13.6	DL + $2\frac{1}{2}$ LL	
		13.8	DL + 3 LL	
		13.9	DL + $3\frac{1}{2}$ LL	
		13.10	DL + 3 3/4 LL	
August 6	14	14.9	DL + $3\frac{1}{2}$ LL	
		14.10	DL + 3 3/4 LL	
		14.11	DL + 4 LL	
		14.12	DL + $4\frac{1}{4}$ LL	
		14.13	DL + $4\frac{1}{2}$ LL	
		14.14	DL + 4 3/4 LL	
		14.15	DL + 5 LL	
		14.16	DL + $5\frac{1}{4}$ LL	
		14.17	DL + $5\frac{1}{2}$ LL	

TABLE VII

PERCENTAGE CHANGE IN CONCRETE STRAINS
DURING AND AFTER 28-DAY TEST

Gauge No.	Concrete Stress At Gauge psi	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
CM 10	420	25	76	85
CM 11	1100	54	63	74
CM 12	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 VIII

END AND CENTRE REACTIONS DURING ASYMMETRICAL LOADINGS

Total Load Required (Tons)	Total Load Measure (Tons)	End Reaction		Load On Span	Centre Support		Load On Span	End Reaction	
		Tons	Per Cent		Tons	Per Cent		Tons	Per Cent
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

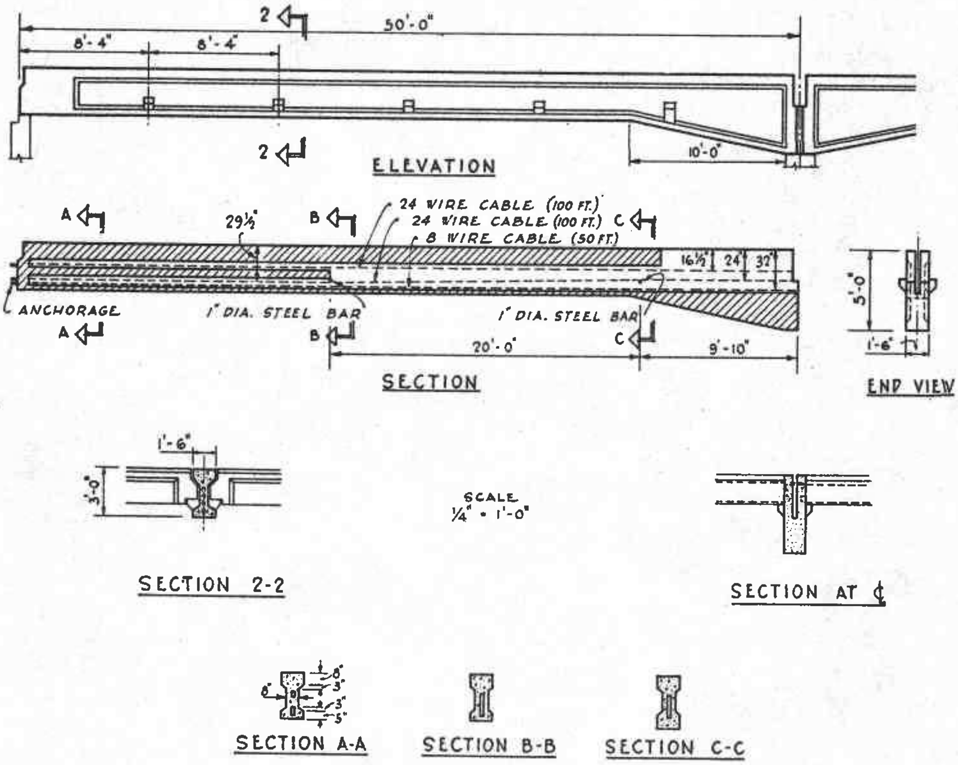


FIGURE 1 DETAILS OF BEAM

BR 3223-1

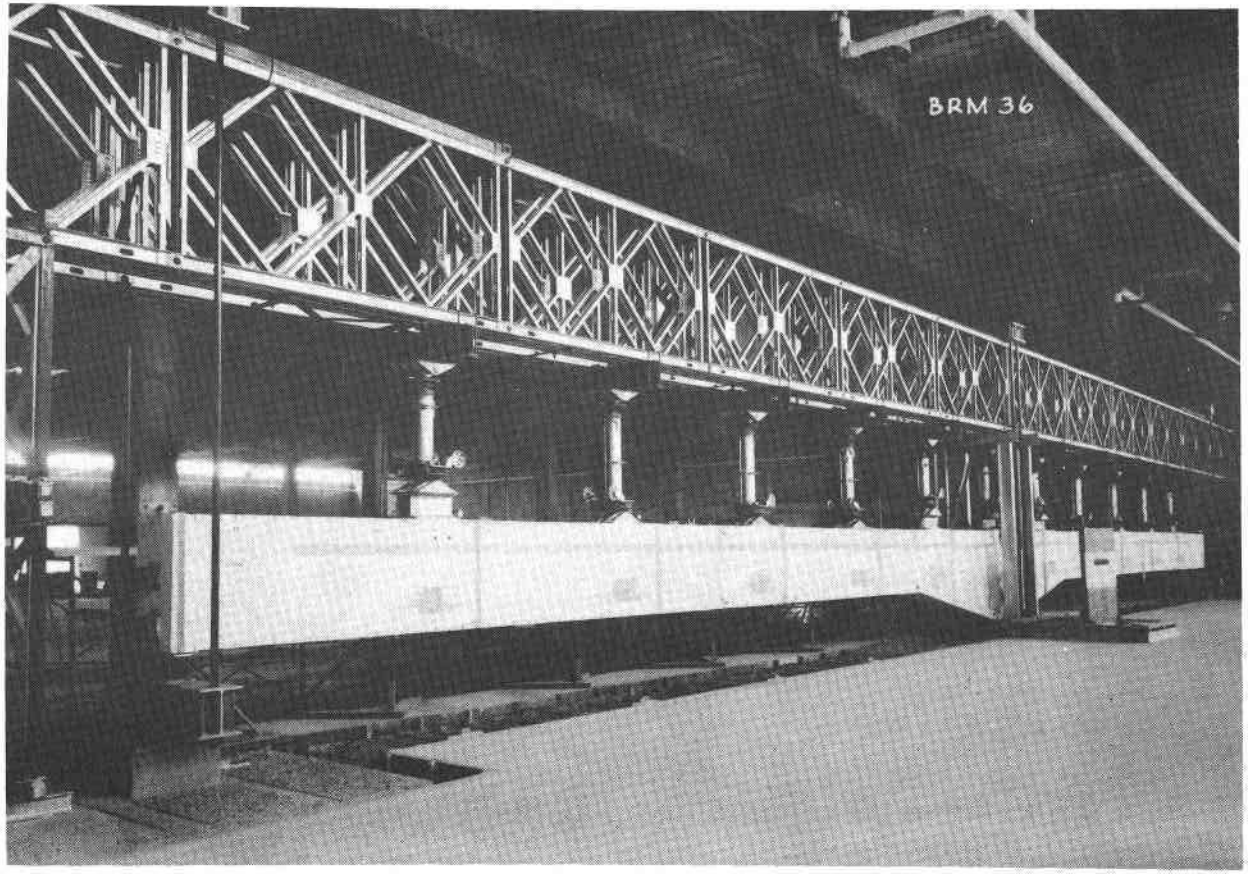


Figure 2(a) Over-all View of Test Set Up.

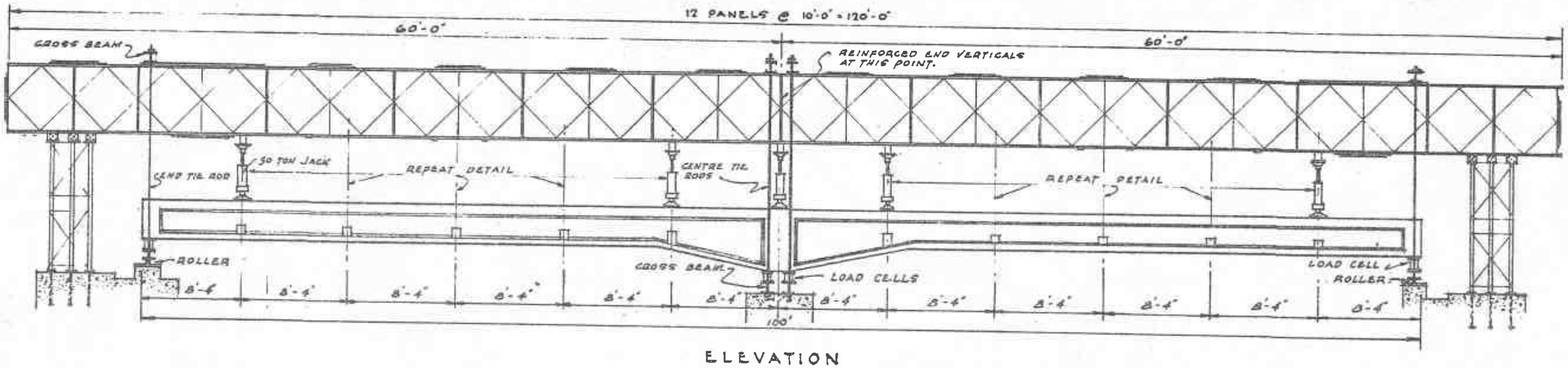


FIGURE 2 (b) TEST APPARATUS

BR 3223-2

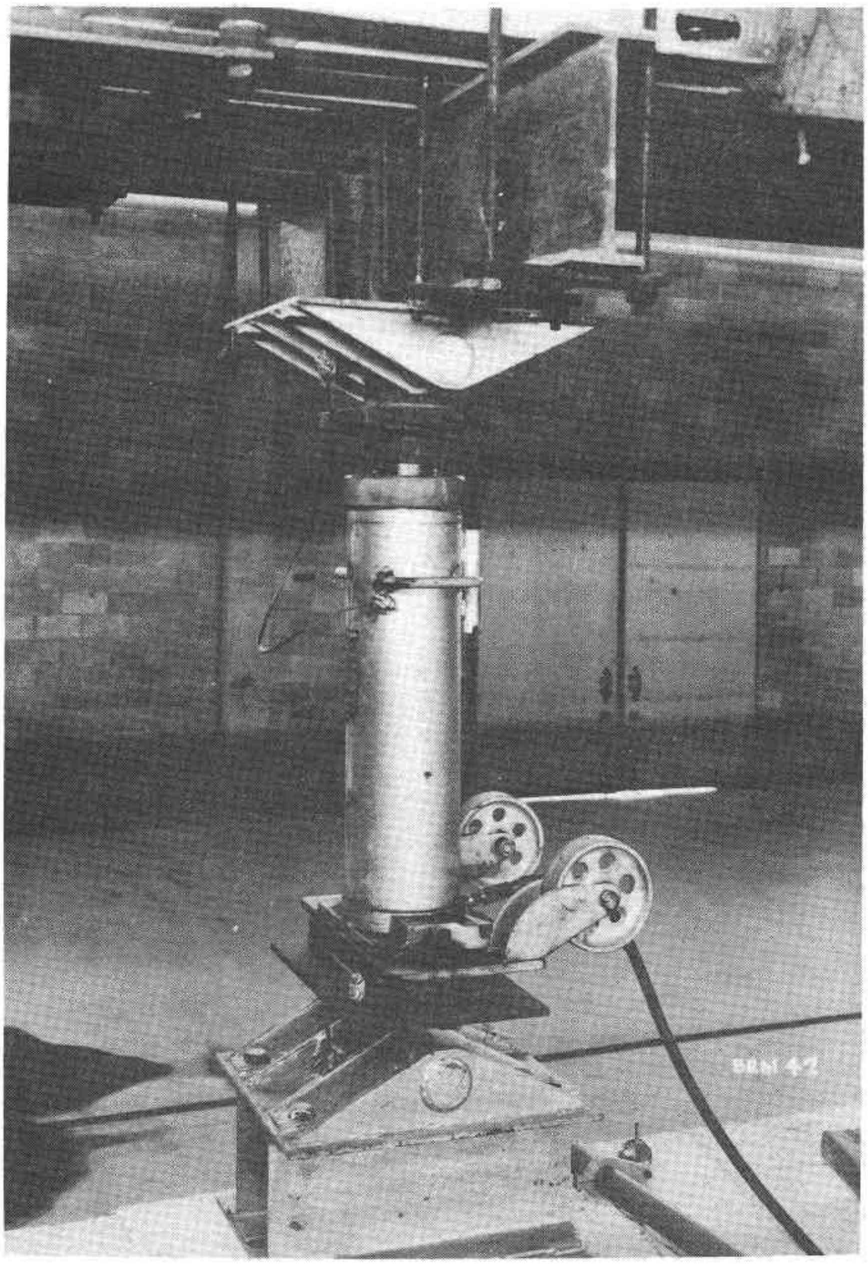


Figure 3 Typical Hydraulic Ram Used for Applying Load.

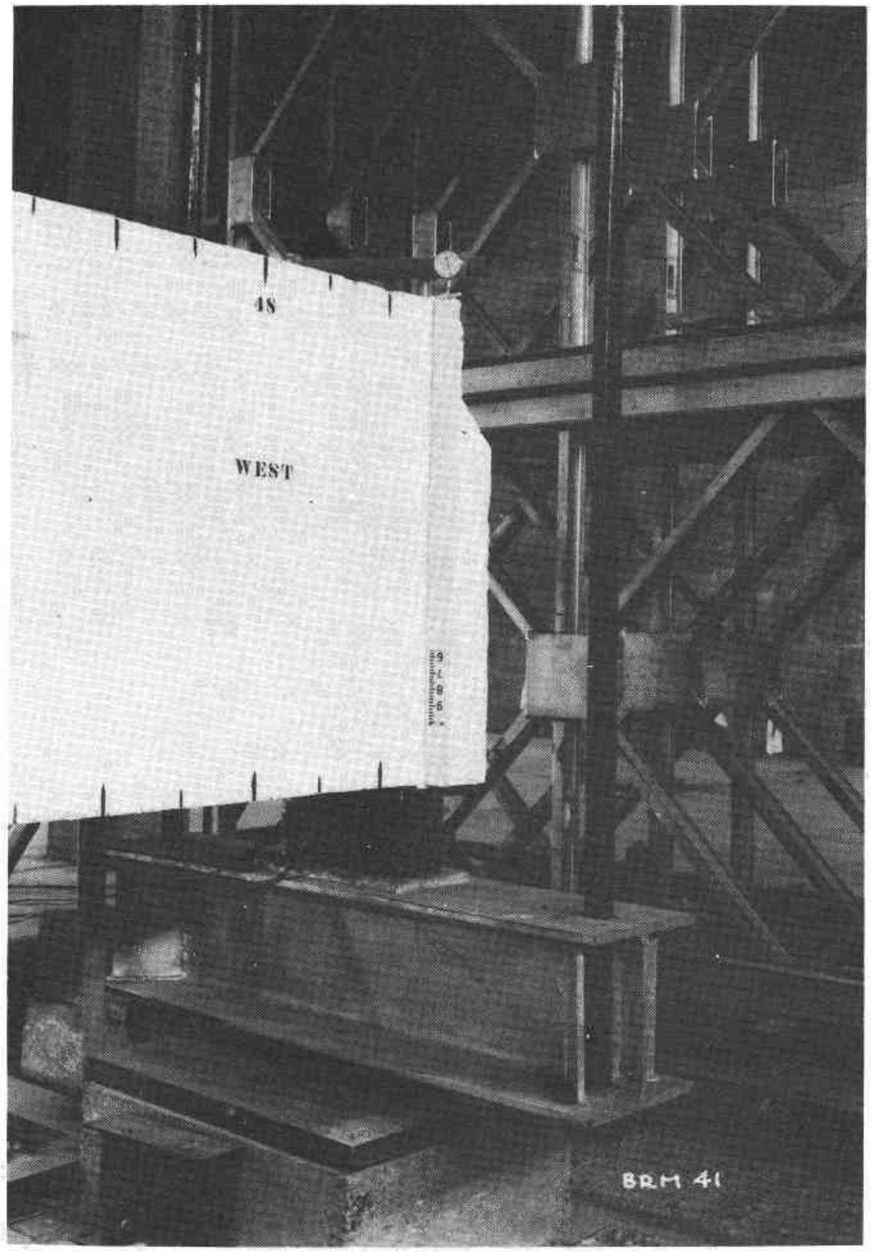


Figure 4 End Reaction of Beam. Note Load Cells and Deflection Apparatus.

Figure 5 Hydraulic Pumps and Gauges Used for Loading.

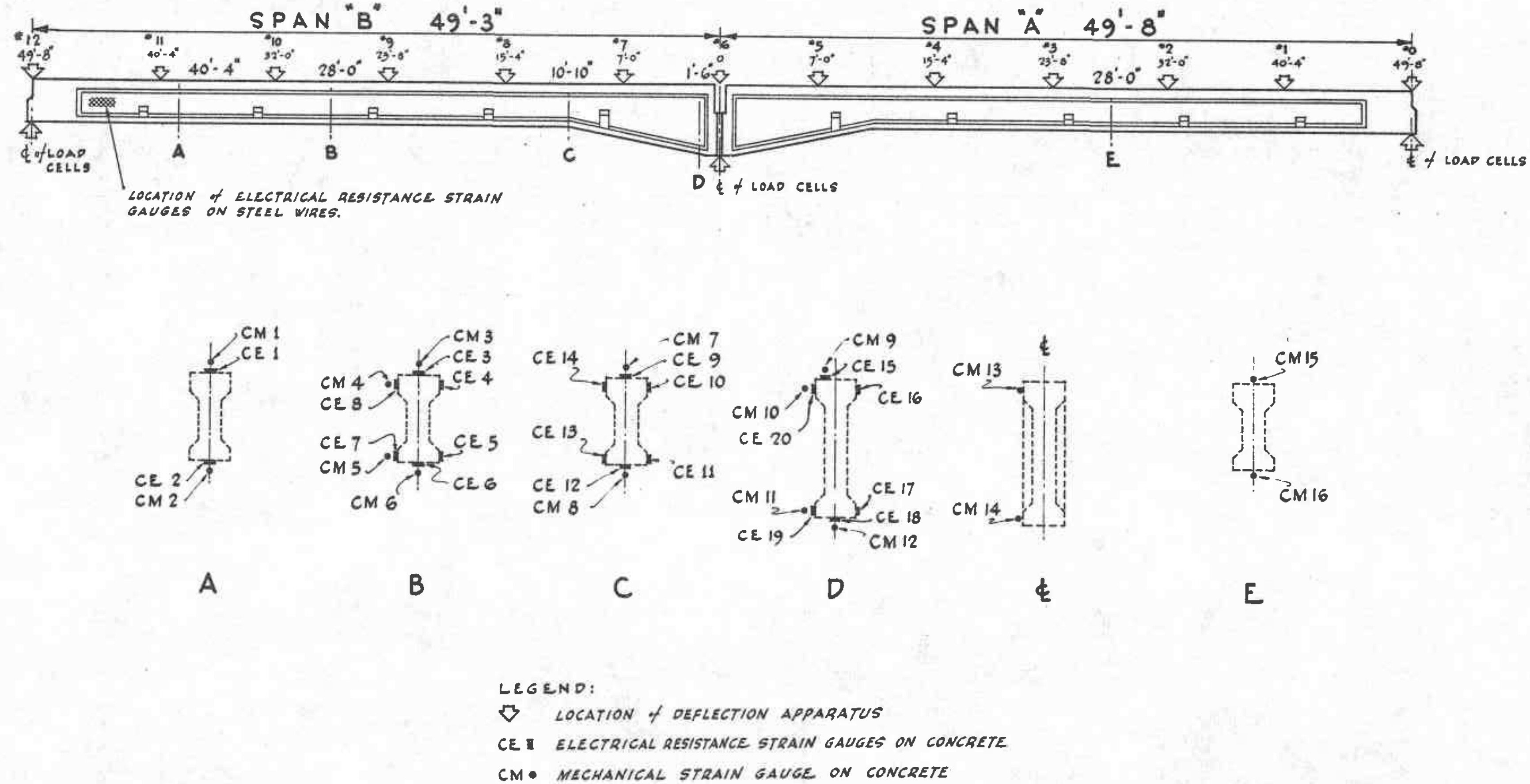
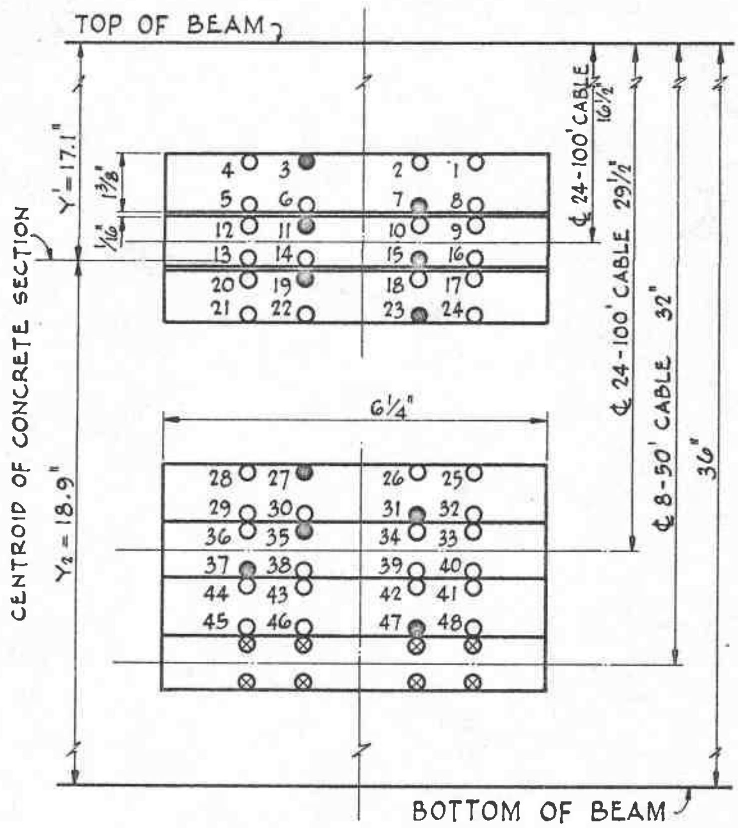


FIGURE 6

LOCATION OF STRAIN AND DEFLECTION GAUGES

BR 3223-3



LEGEND: ● WIRES WITH ELECTRICAL RESISTANCE STRAIN GAUGES
⊗ 50' WIRES (TENSIONED ON CASTING BED)

THE NUMBERS 1 TO 48 INDICATE SEQUENCE of TENSIONING

FIGURE 7

LOCATION OF GAUGED PRESTRESSING WIRES

BR 3223-4

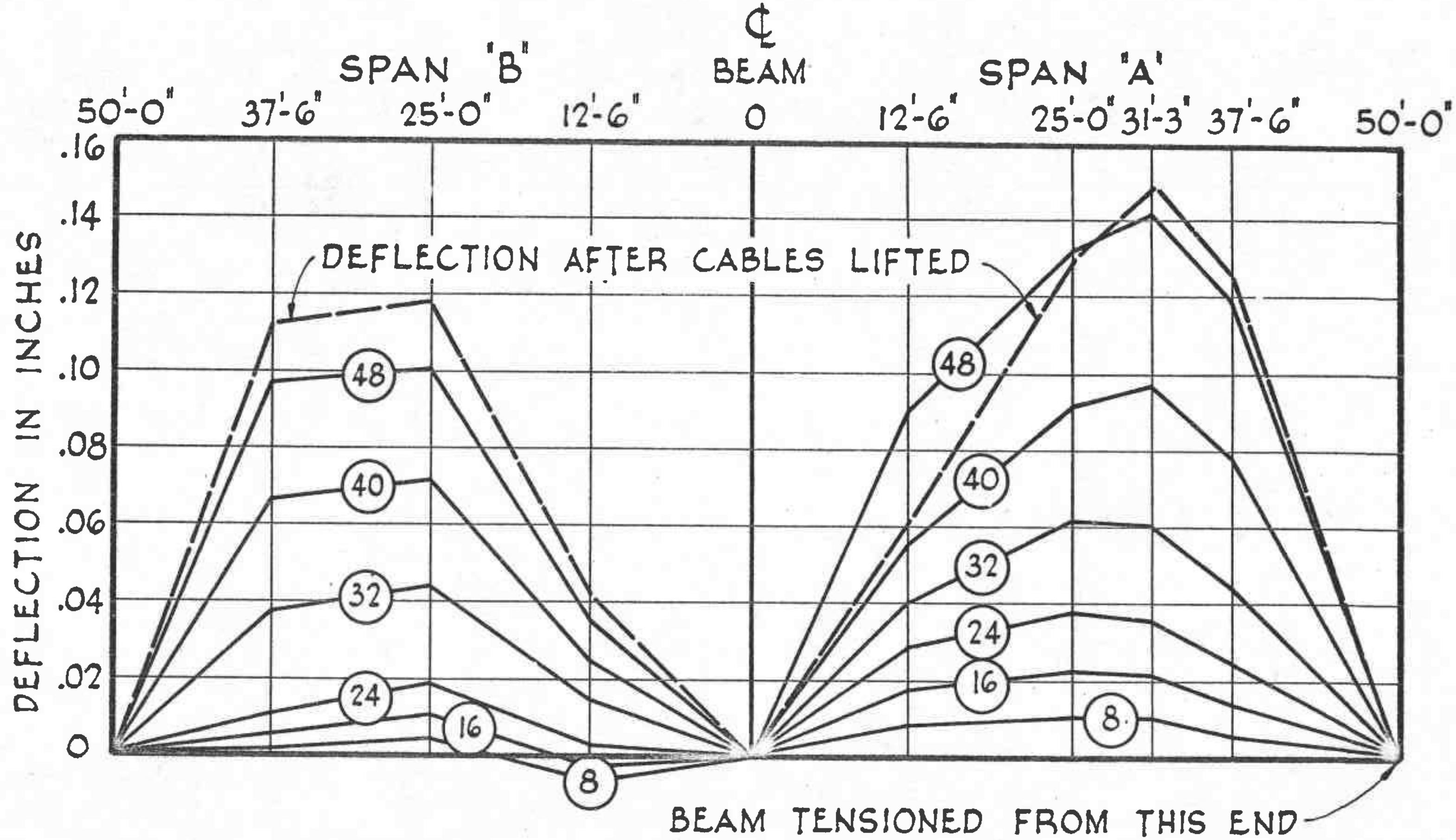


FIGURE 8

HISTORY OF DEFLECTION CHANGES DURING PRESTRESSING OF THE FORTY-EIGHT 100-FT WIRES

BR 3223-5

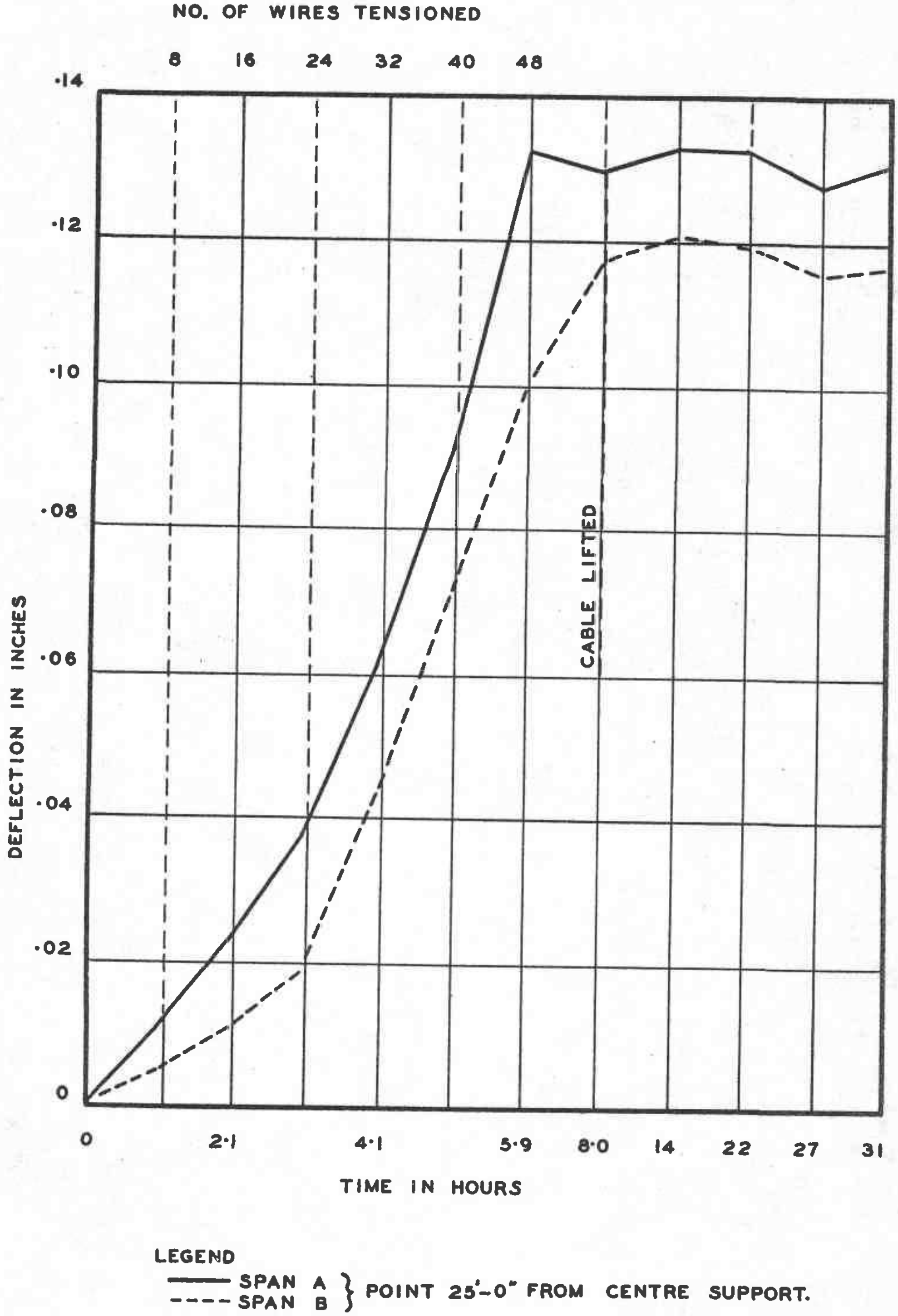


FIGURE 9

DEFLECTION OF TWO SIMILAR POINTS ON SPANS A AND B DURING PRESTRESSING

BR 3223-6

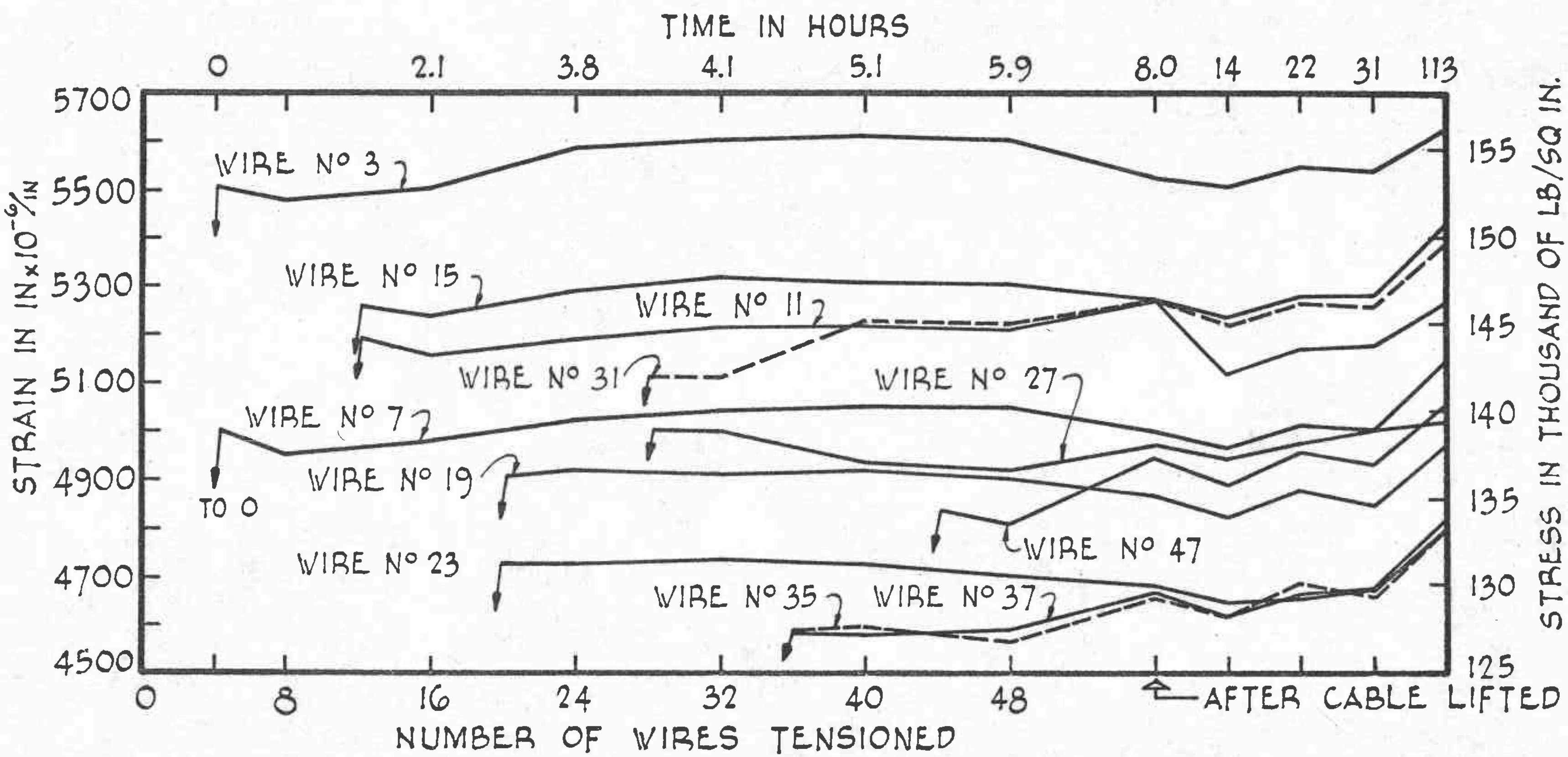


FIGURE 10

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

BR3223-7

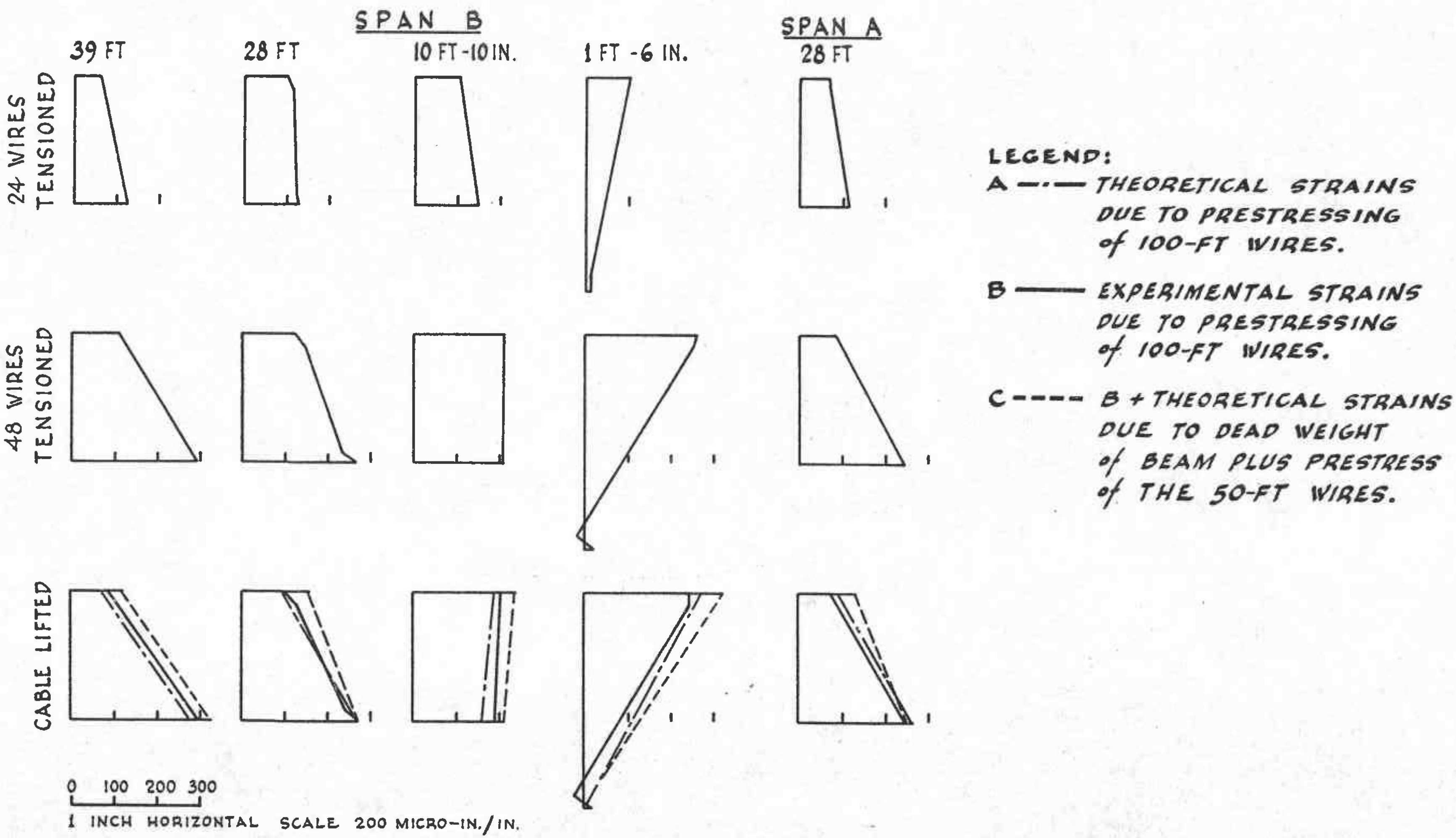


FIGURE 11

CONCRETE STRAIN CHANGES DUE TO PRESTRESSING OF 100-FT WIRES

BR 3223-8

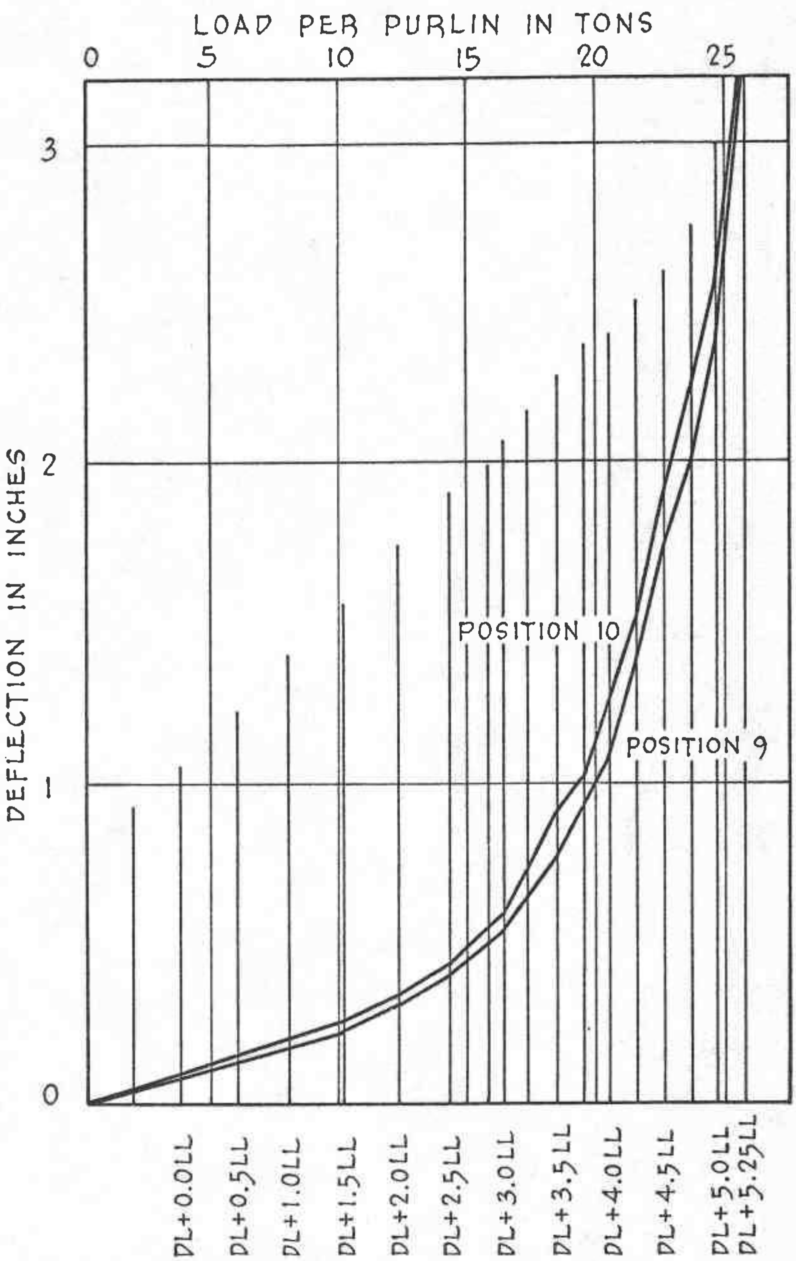


FIGURE 12

LOAD-DEFLECTION CURVES FOR DEFLECTION GAUGE POSITIONS 9 AND 10

Deflection Values from the First Application of Load

BR3223-9

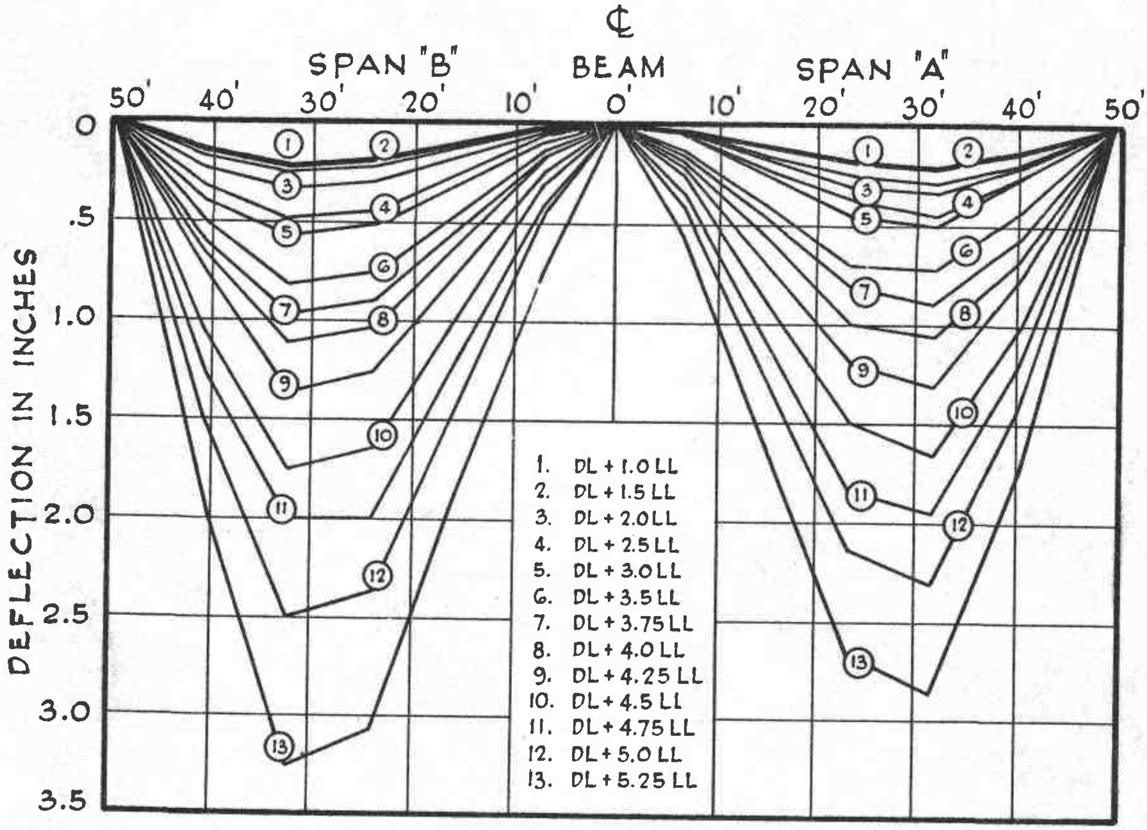


FIGURE 13

DEFLECTIONS OF BEAM UP TO FINAL LOAD
Data from Wire and Pulley System

BR 3223 - 10

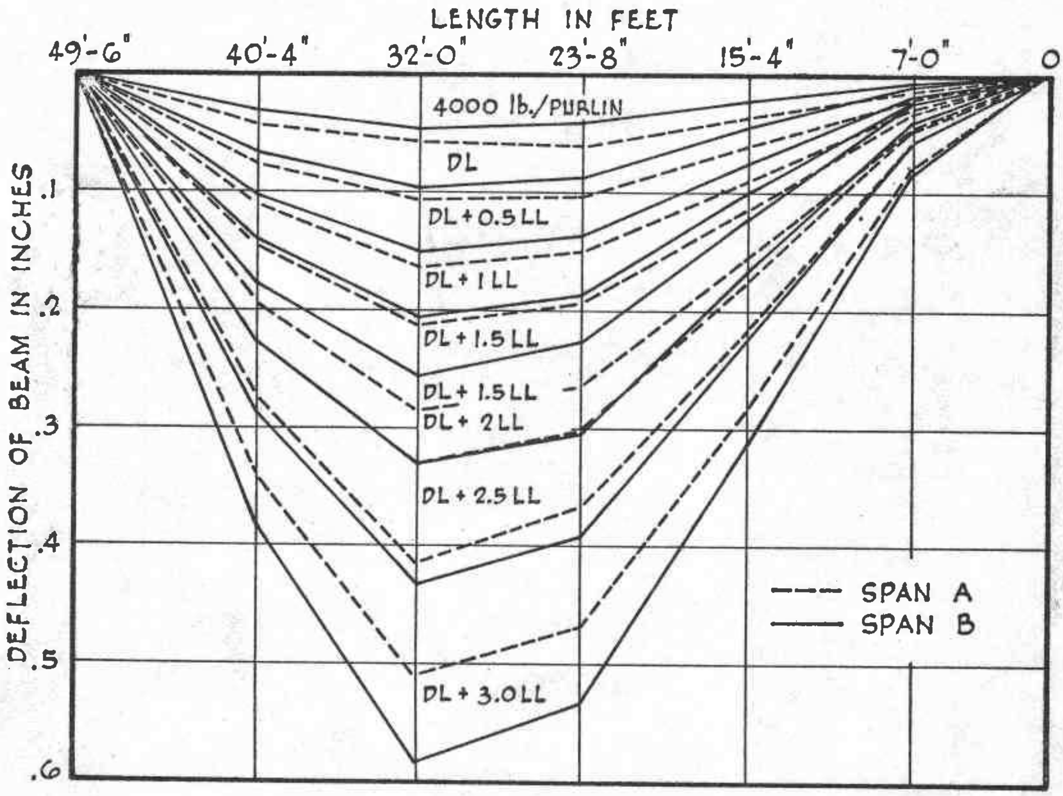


FIGURE 14

SPAN A AND B DEFLECTIONS FROM 0 TO 3 LL + DL
Data from Dial Gauges

BR 3223-11

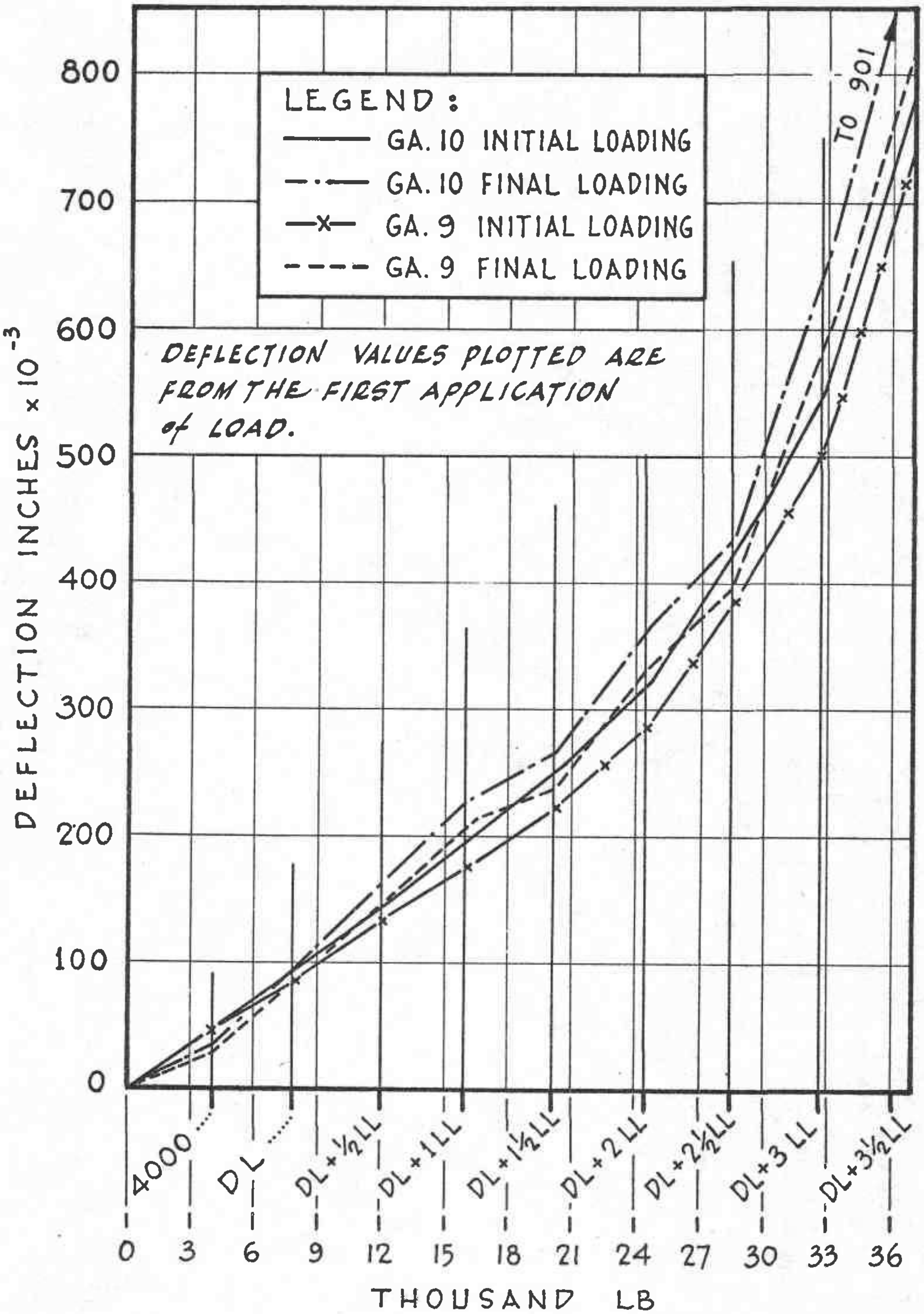


FIGURE 15

INITIAL AND FINAL DEFLECTIONS MEASURED AT VARIOUS APPLIED LOADS

BR 3223-12

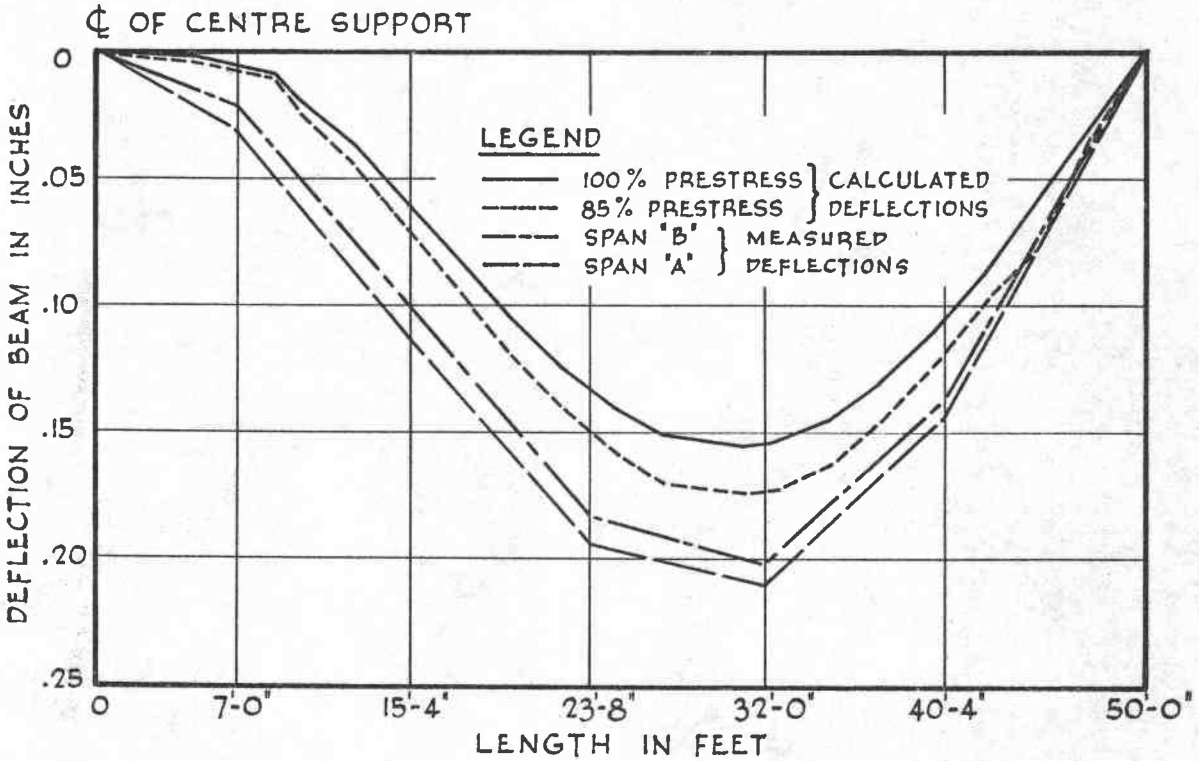


FIGURE 16

CALCULATED AND MEASURED DEFLECTIONS FOR DEAD LOAD PLUS 1 LIVE LOAD

BR 3223-13

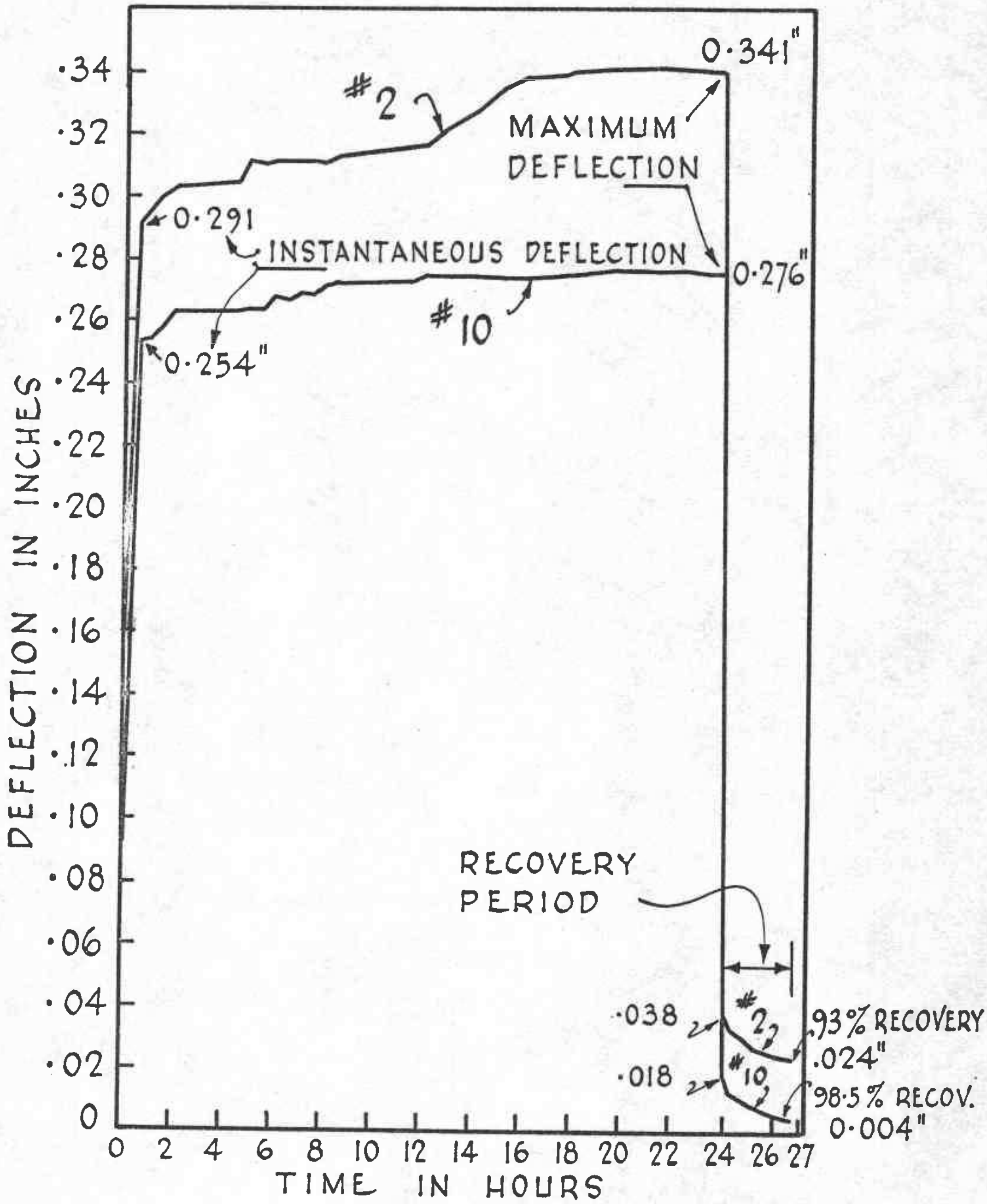


FIGURE 17

DEFLECTION OF BEAM AT GAUGES 2 AND 10
DURING NATIONAL BUILDING CODE TEST
(DL + 1½ LL)

BR 3223-14

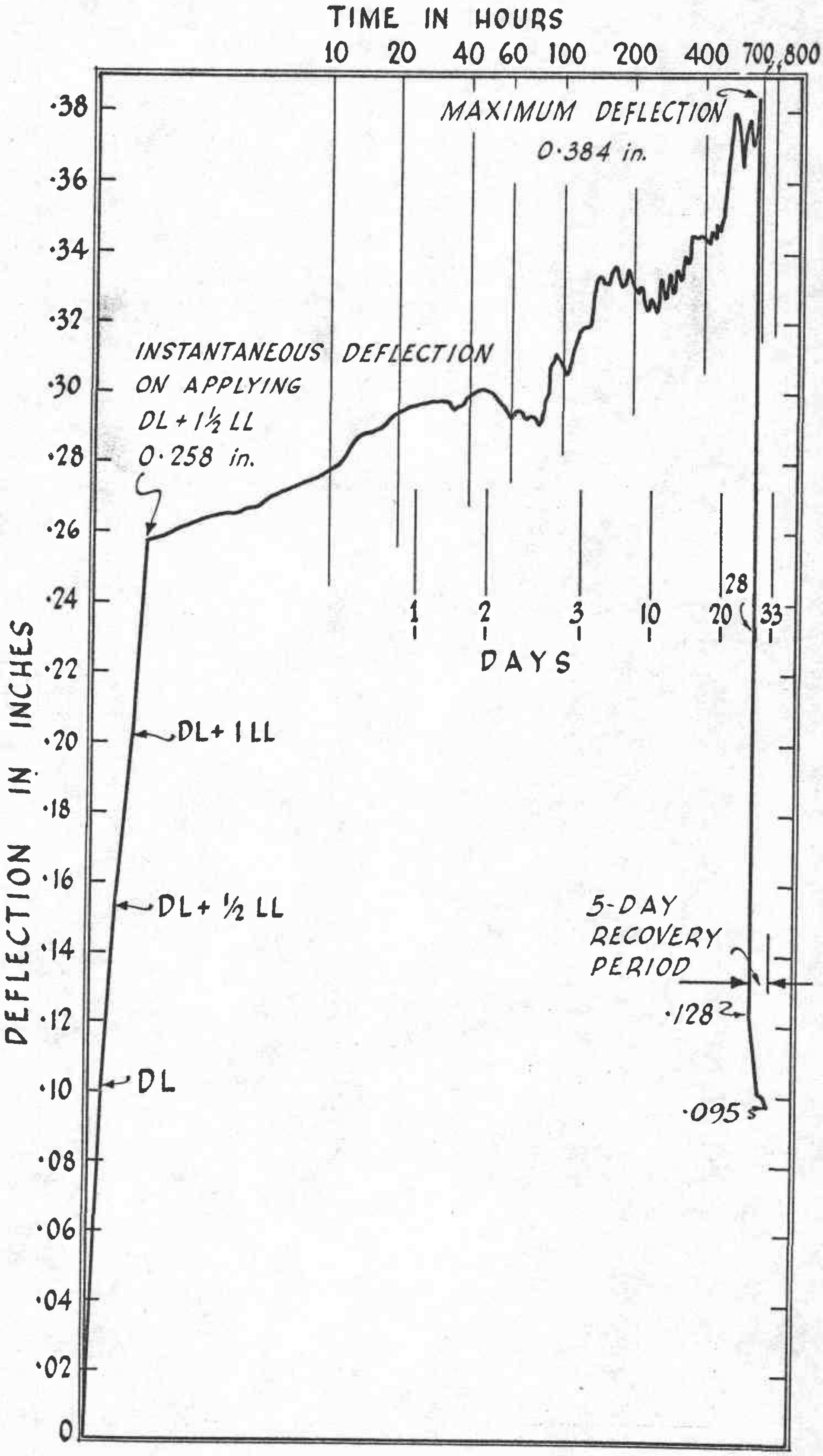


FIGURE 18

DEFLECTION OF BEAM AT GAUGE NO. 10
DURING AND AFTER 28-DAY TEST

BR 3223-15

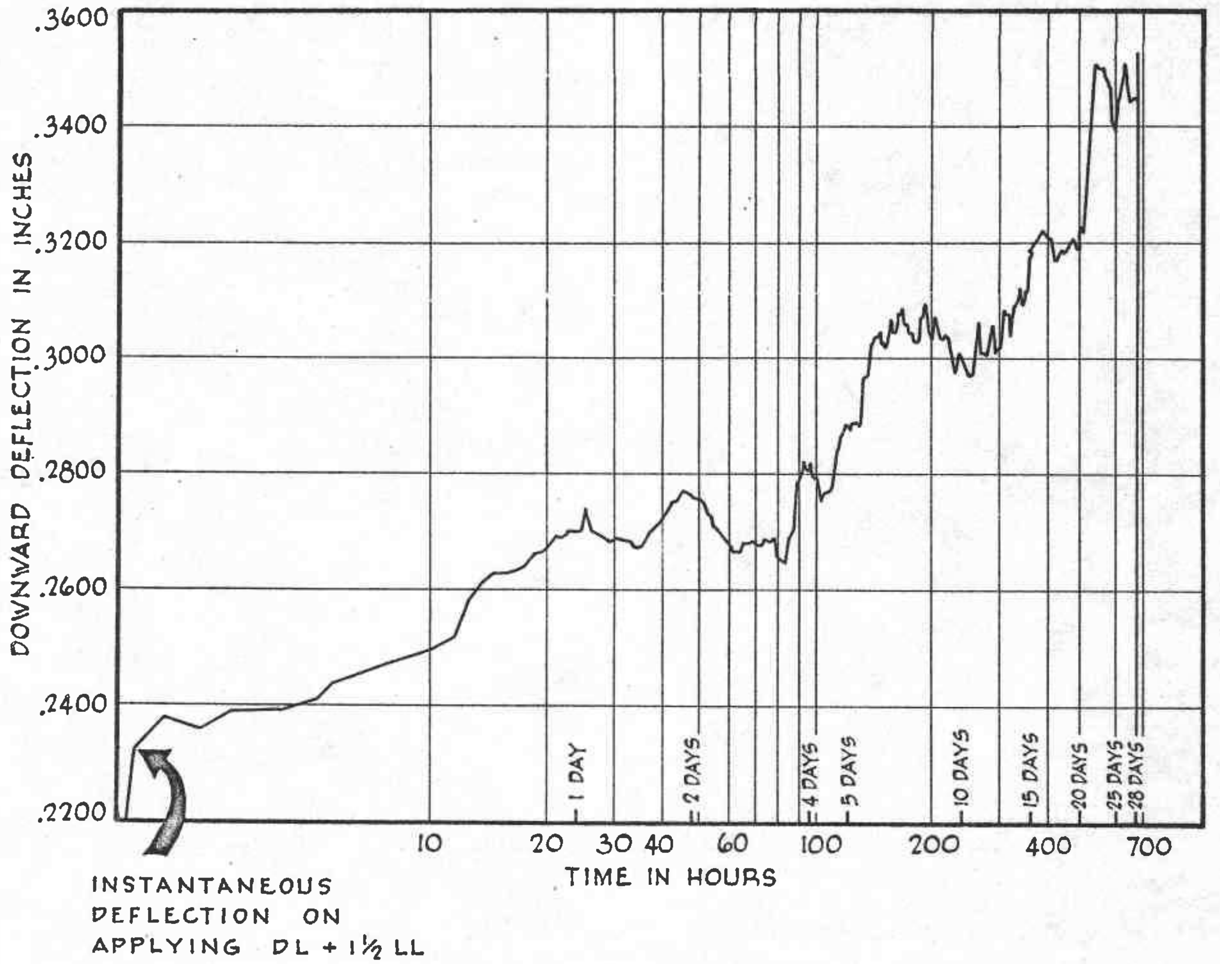


FIGURE 19

DEFLECTION OF BEAM AT GAUGE NO. 9 DURING 28-DAY TEST

BR 3223-16

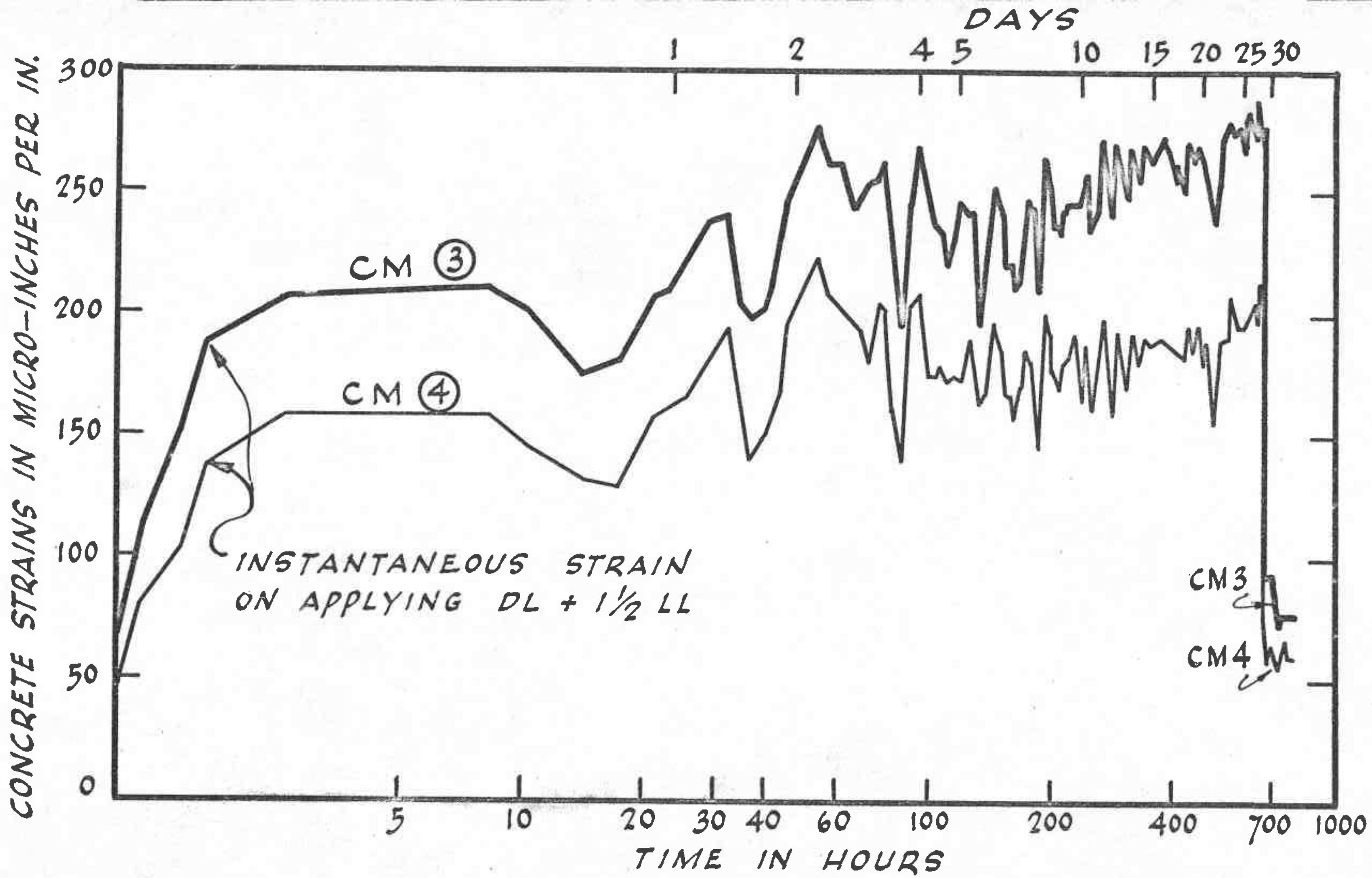


FIGURE 20

HISTORY OF CONCRETE COMPRESSIVE STRAINS AT POSITIONS CM 3 & CM 4 DURING 28 - DAY TEST

BR 3223-17

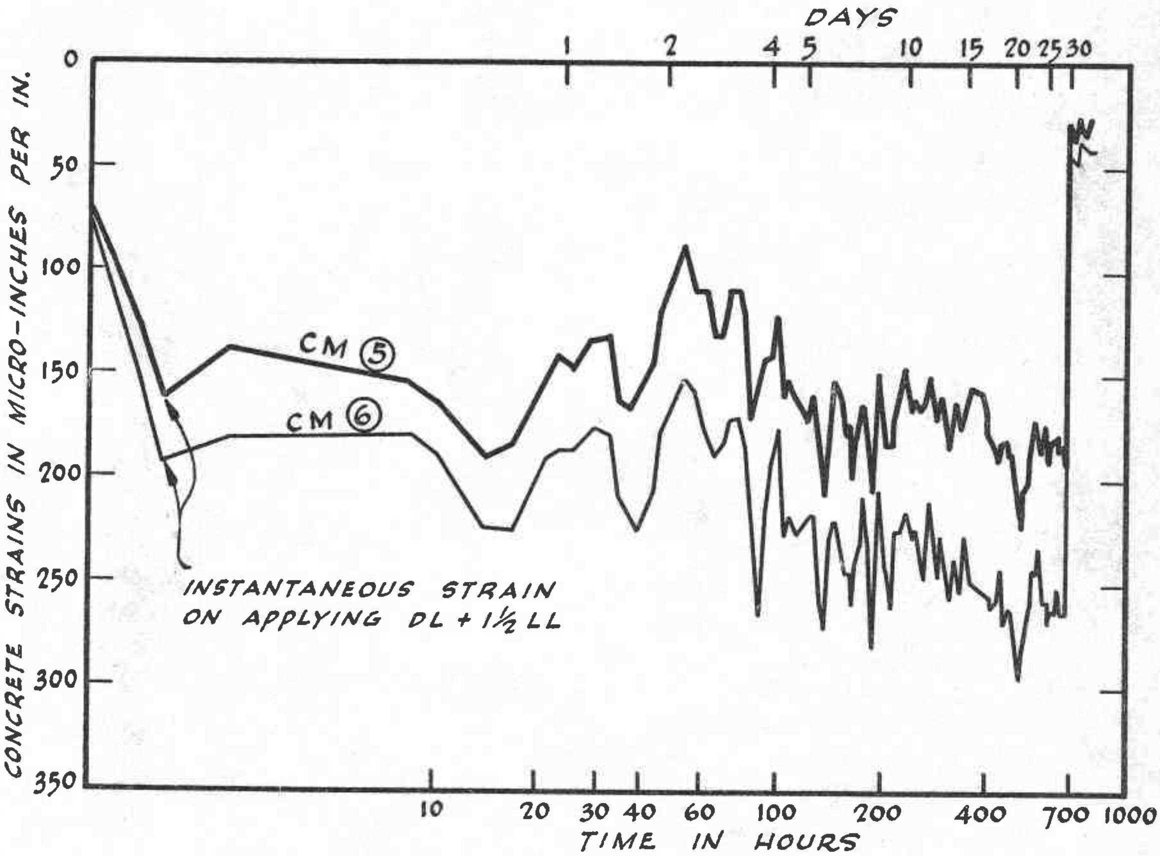


FIGURE 21

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

8R 3223-18

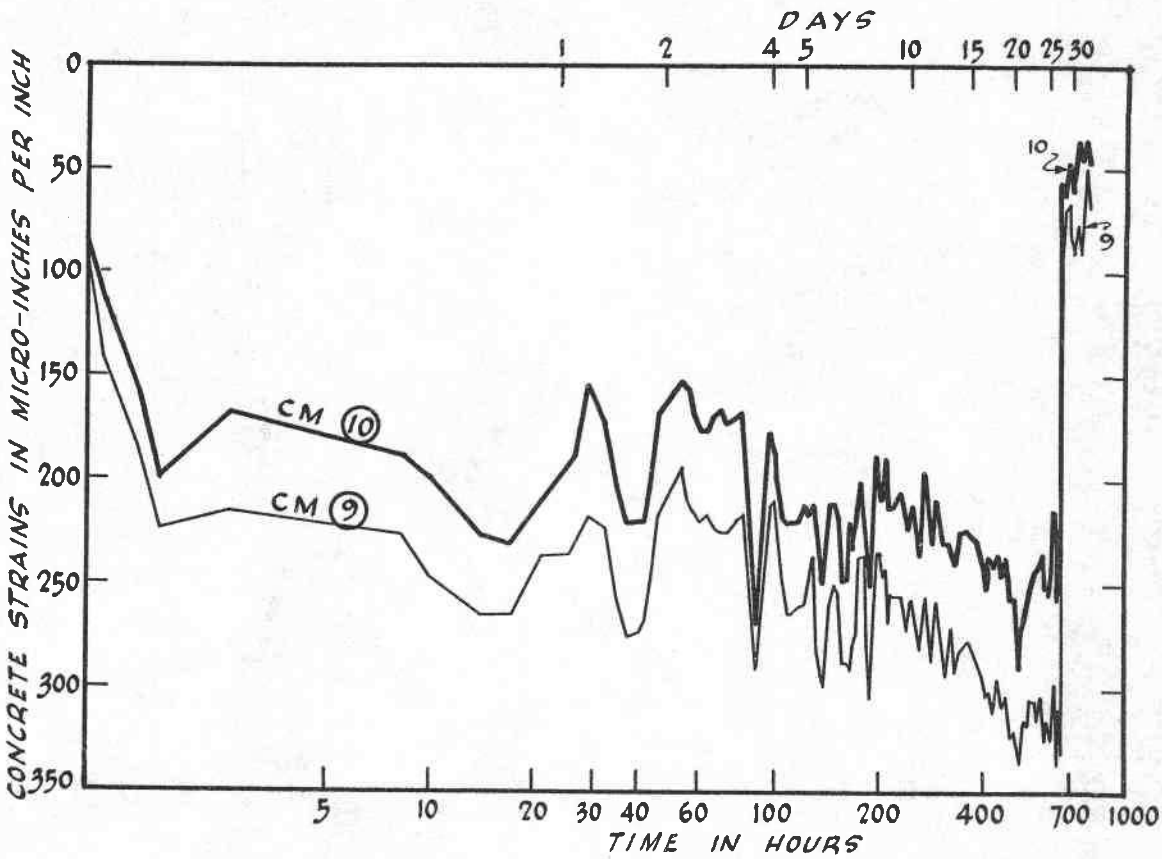


FIGURE 22

HISTORY OF CONCRETE TENSILE STRAINS AT POSITIONS CM 9 & CM 10 DURING 28-DAY TEST

BR 3223-19

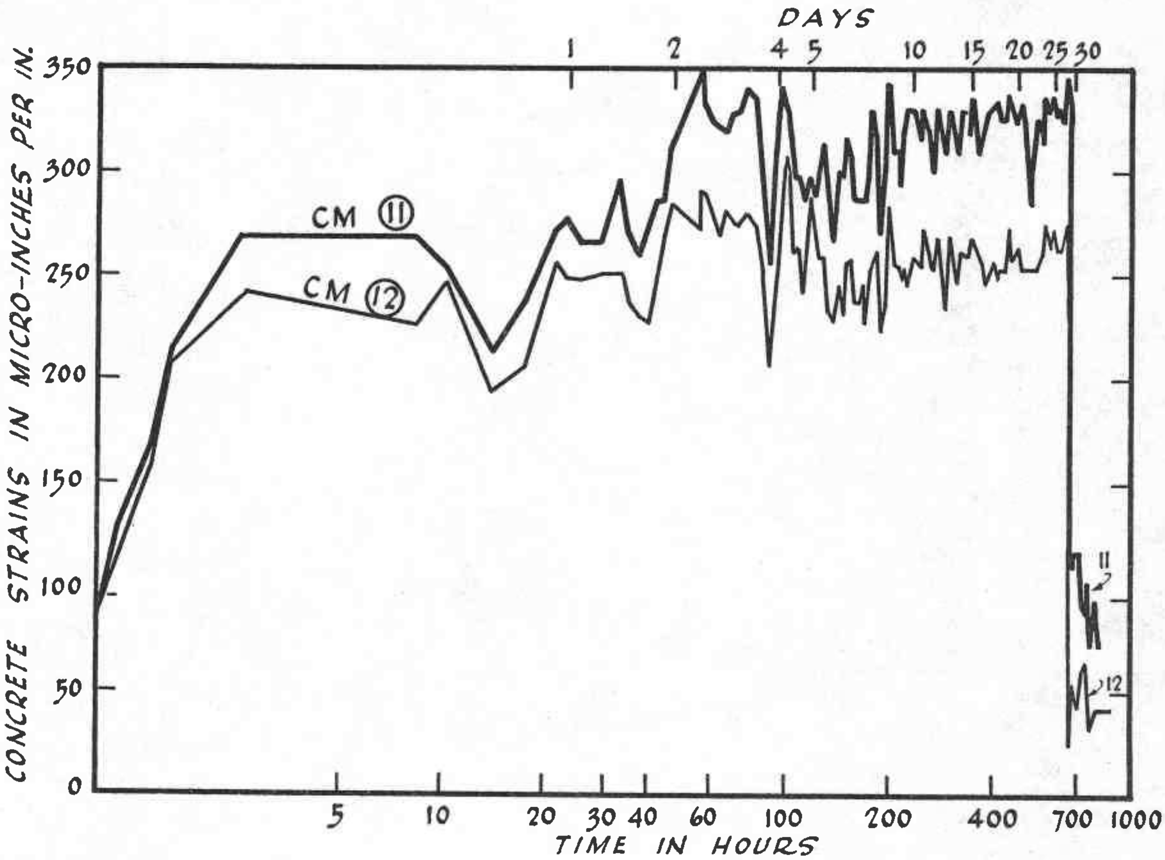


FIGURE 23

HISTORY OF CONCRETE COMPRESSIVE STRAINS AT POSITIONS CM 11 AND CM 12 DURING THE 28-DAY TEST

82 32 23-20

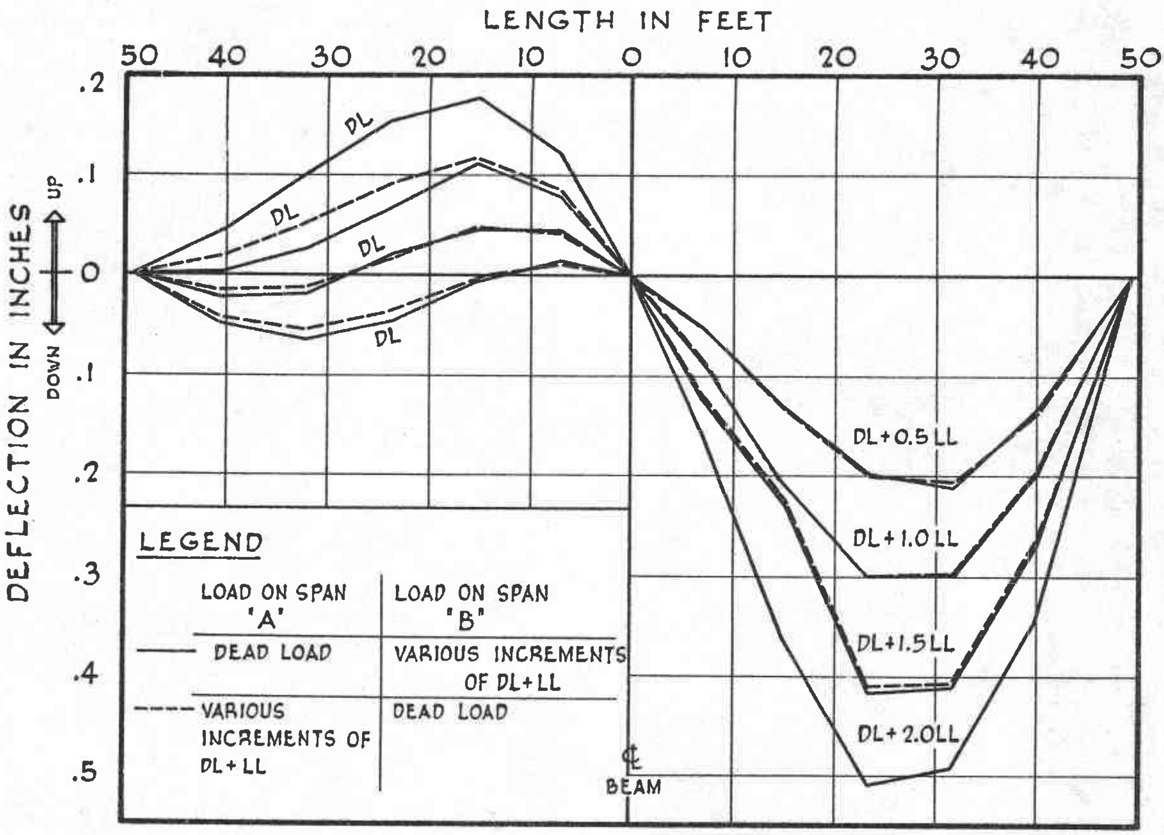


FIGURE 24

DEFLECTIONS OF BEAM DURING ASYMMETRICAL LOADING

BR 3223-21

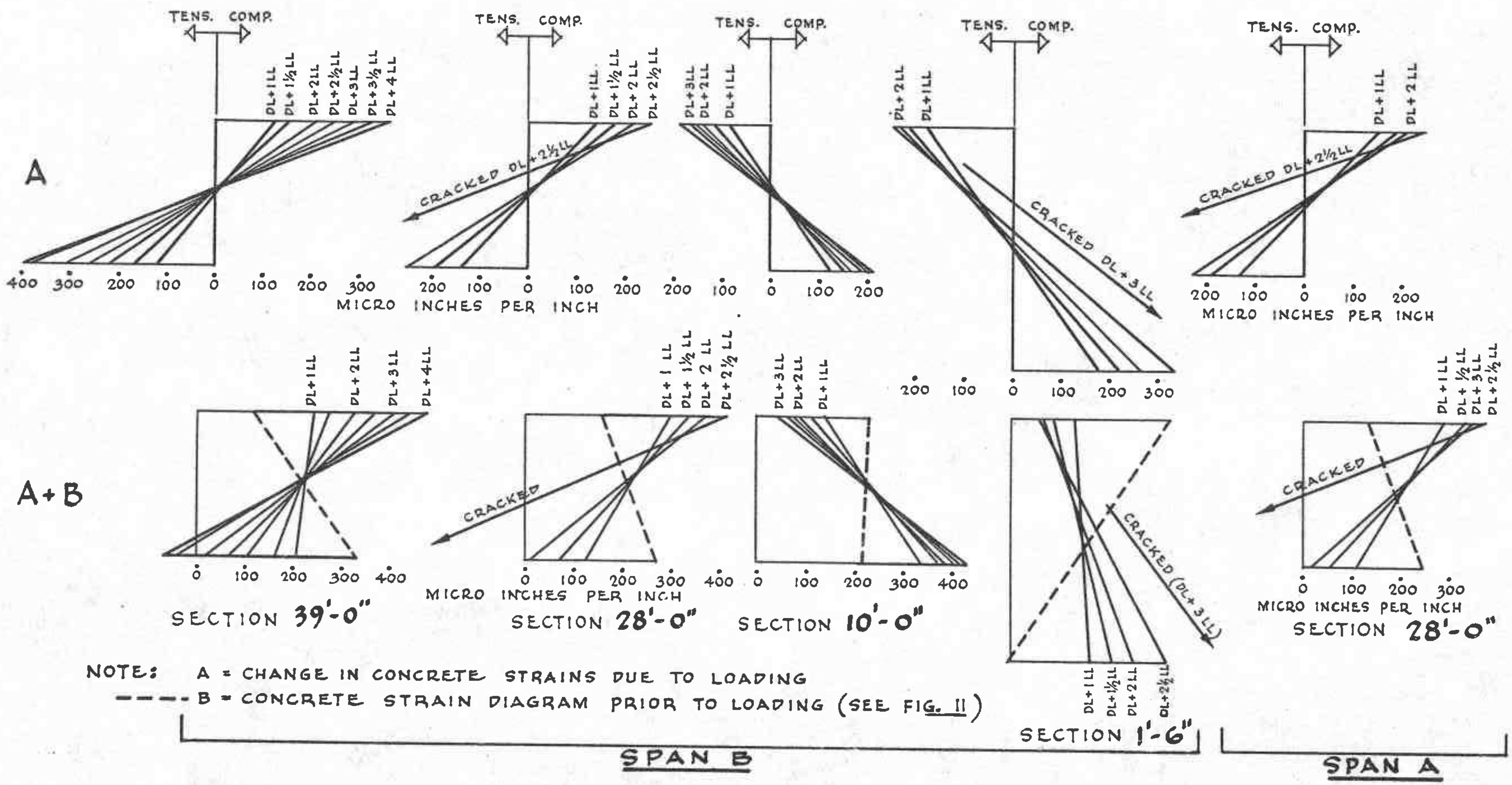


FIGURE 25

CONCRETE STRAINS FUE TO SYMMETRICAL LOADING

BR 3223-22

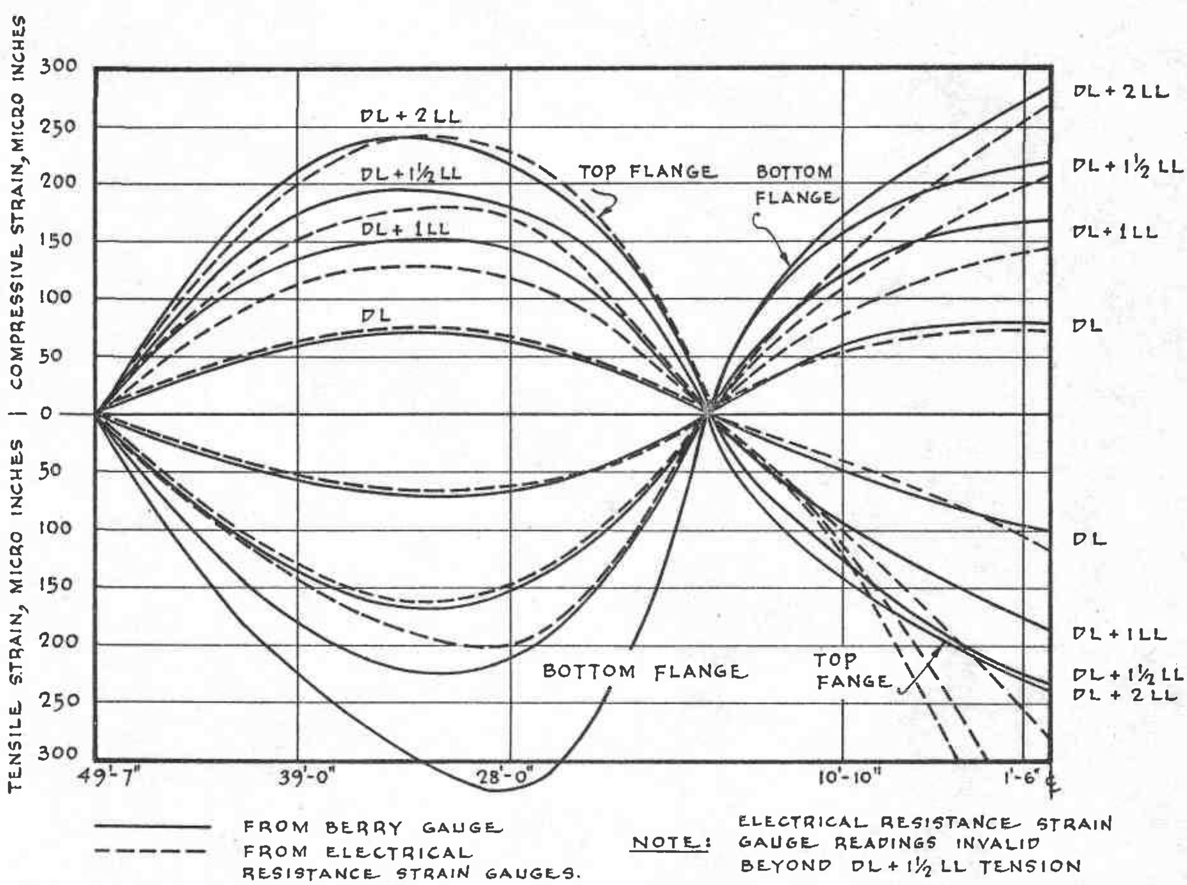
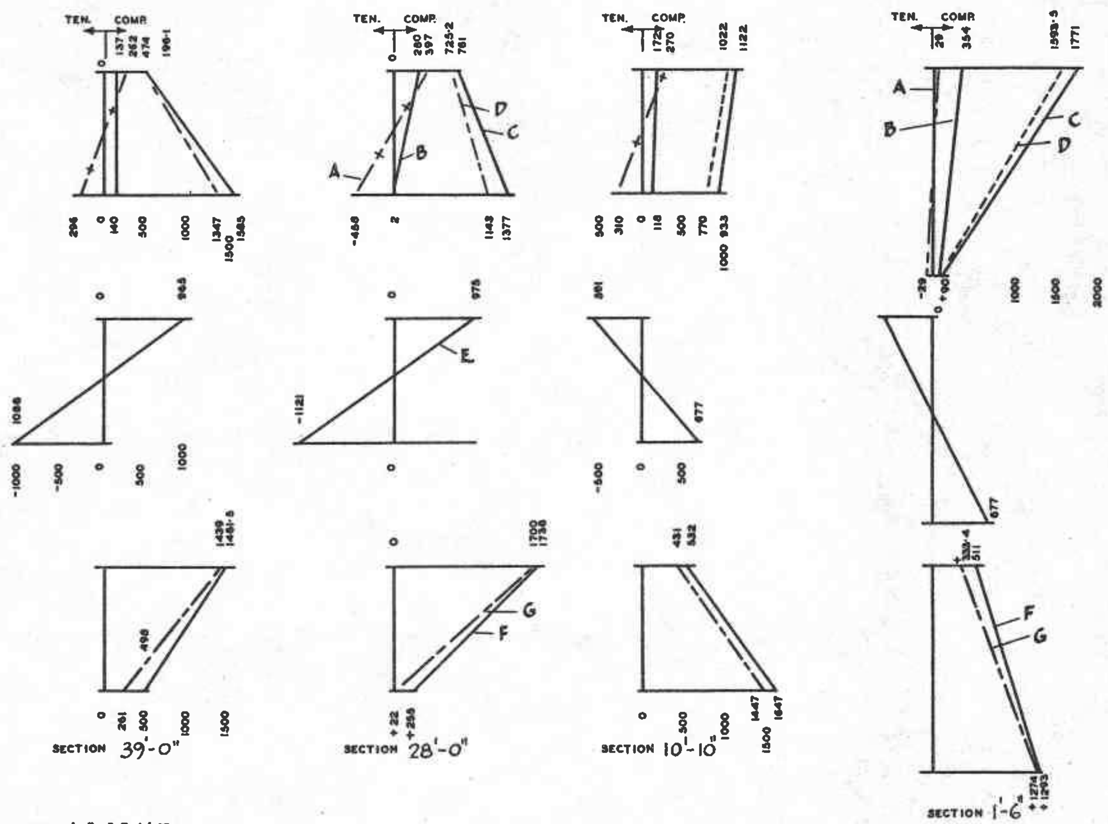


FIGURE 26

CONCRETE STRAINS IN SPAN "B"

BR 3223-23



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

FIGURE 27 THEORETICAL STRESSES

BR 3223-24

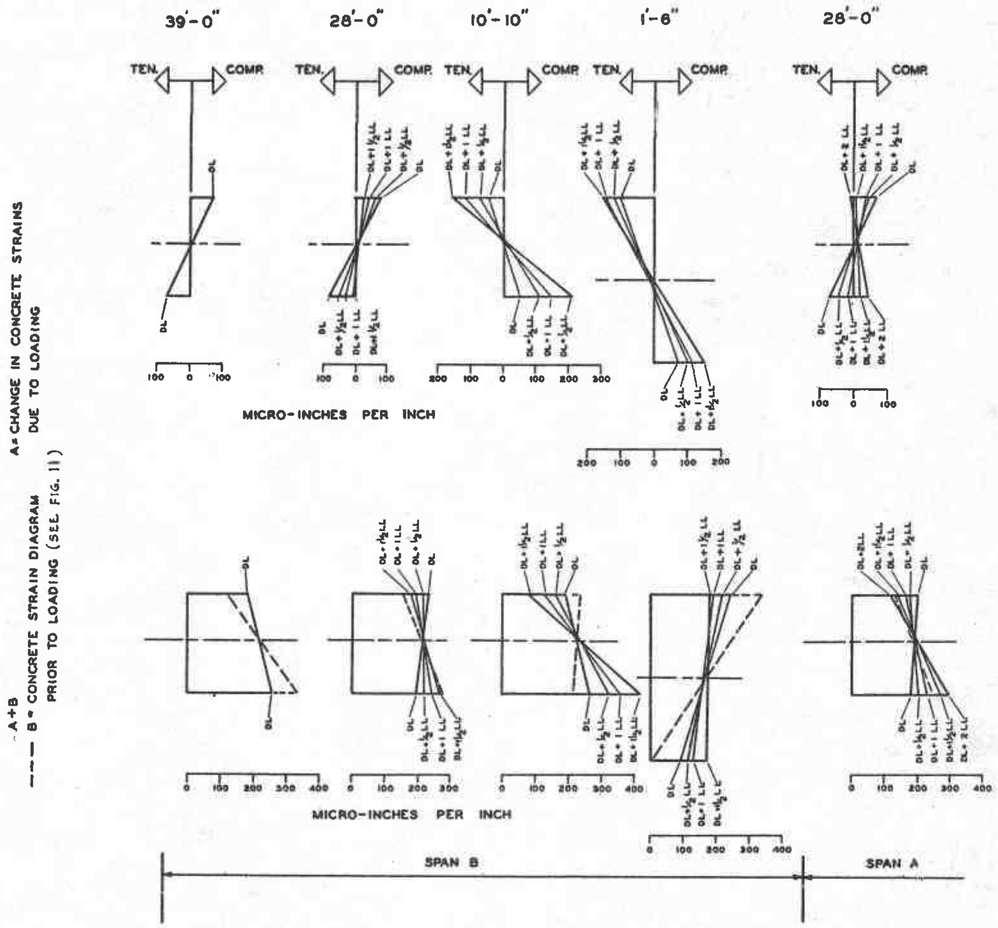


FIGURE 28

MEASURED CONCRETE STRAINS DUE TO ASYMMETRICAL LOADING
Concrete Strains in Span B Loaded with Constant Dead Load of Roof Structure and with DL Plus Various Increments of LL on Span A

0a 3223-25

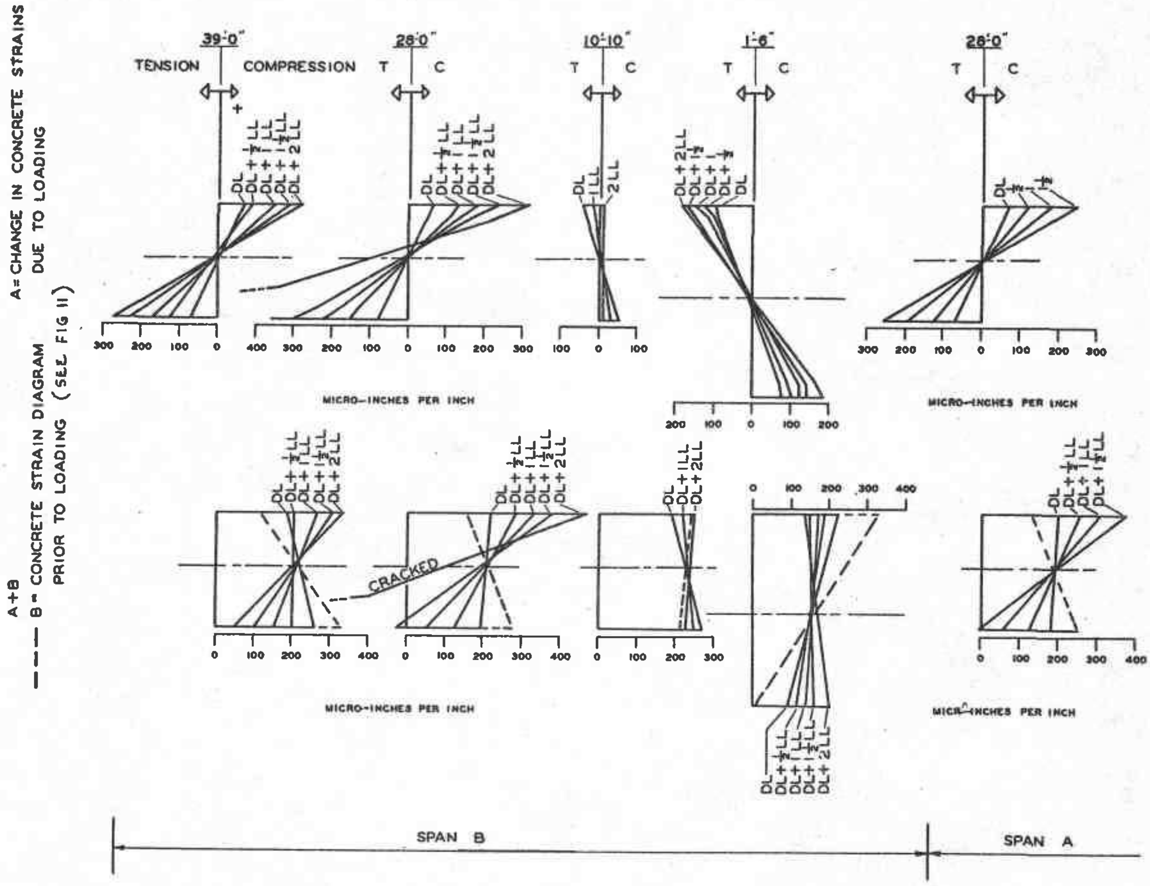


FIGURE 29

MEASURED CONCRETE STRAINS DUE TO ASYMMETRICAL LOADING
Concrete Strains in Span B Loaded with Dead Load Plus Increments of Live Load (Span A Loaded with Dead Load Only)

8R3229-26

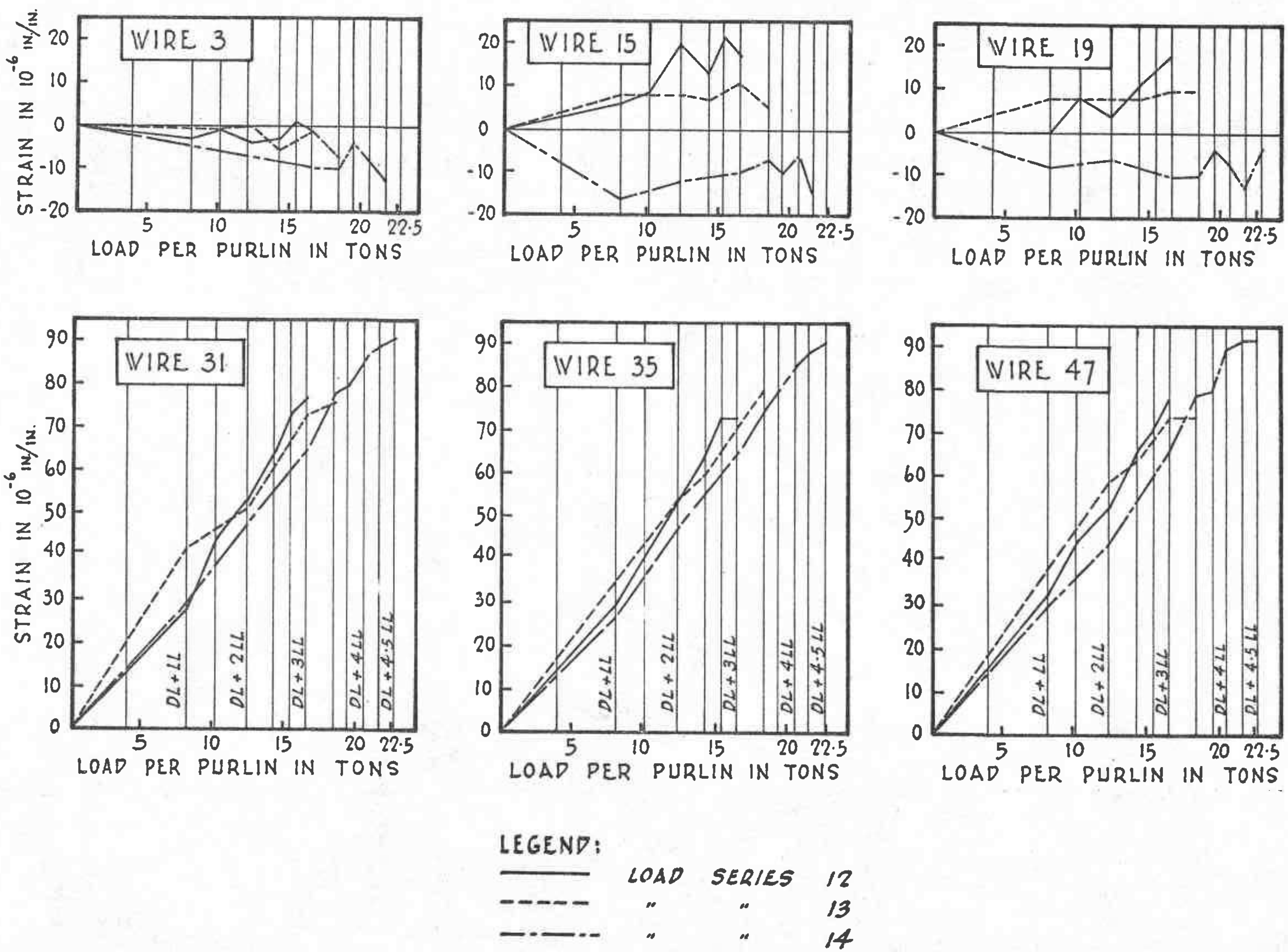
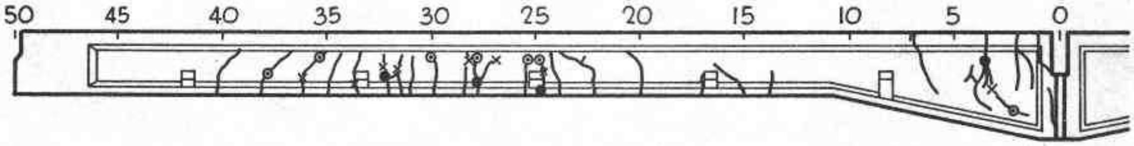


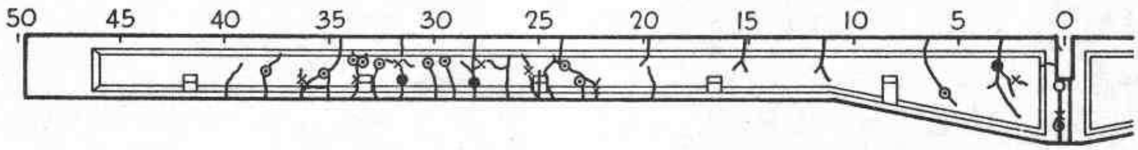
FIGURE 30

HISTORY OF CHANGES IN STRAIN OF SIX WIRES DURING LAST THREE LOAD SERIES

BR 3223-27



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 31

FINAL CRACKING PATTERN OF BEAM

BR 3223-28

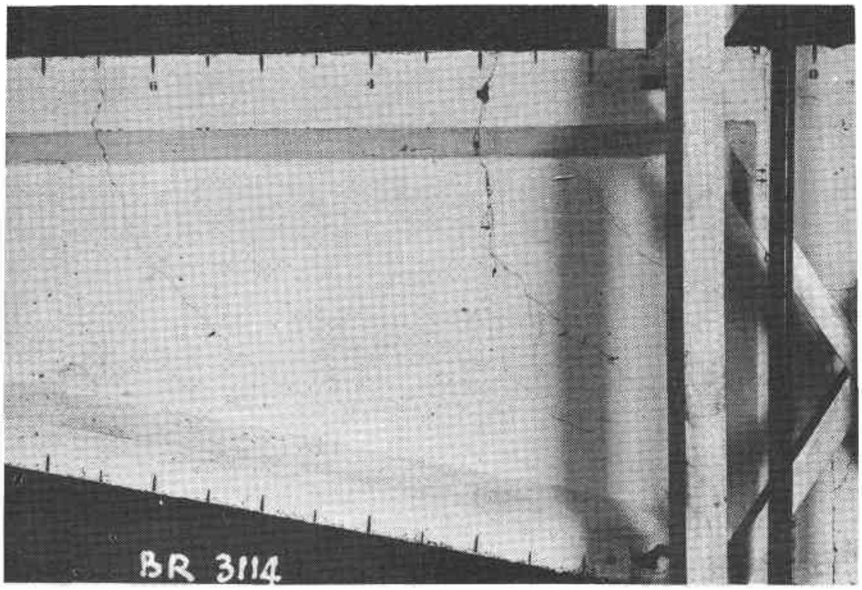


Figure 32 Cracking at Haunch End of Span B

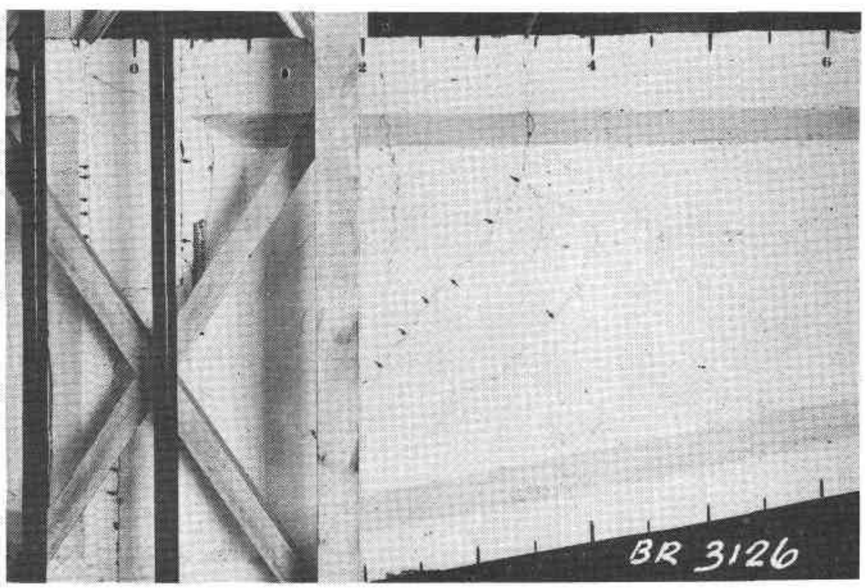


Figure 33 Cracking at Haunch End of Span A

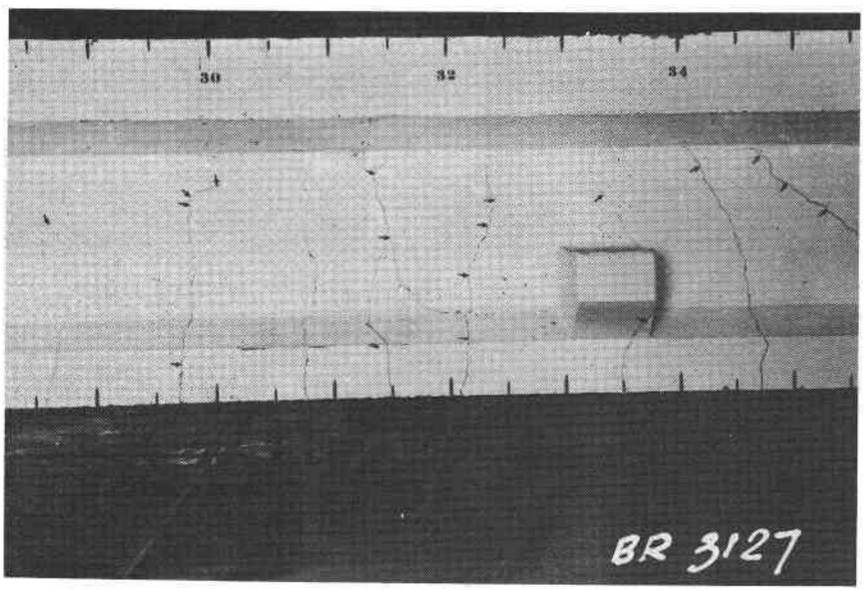


Figure 34 Cracks in Span A.

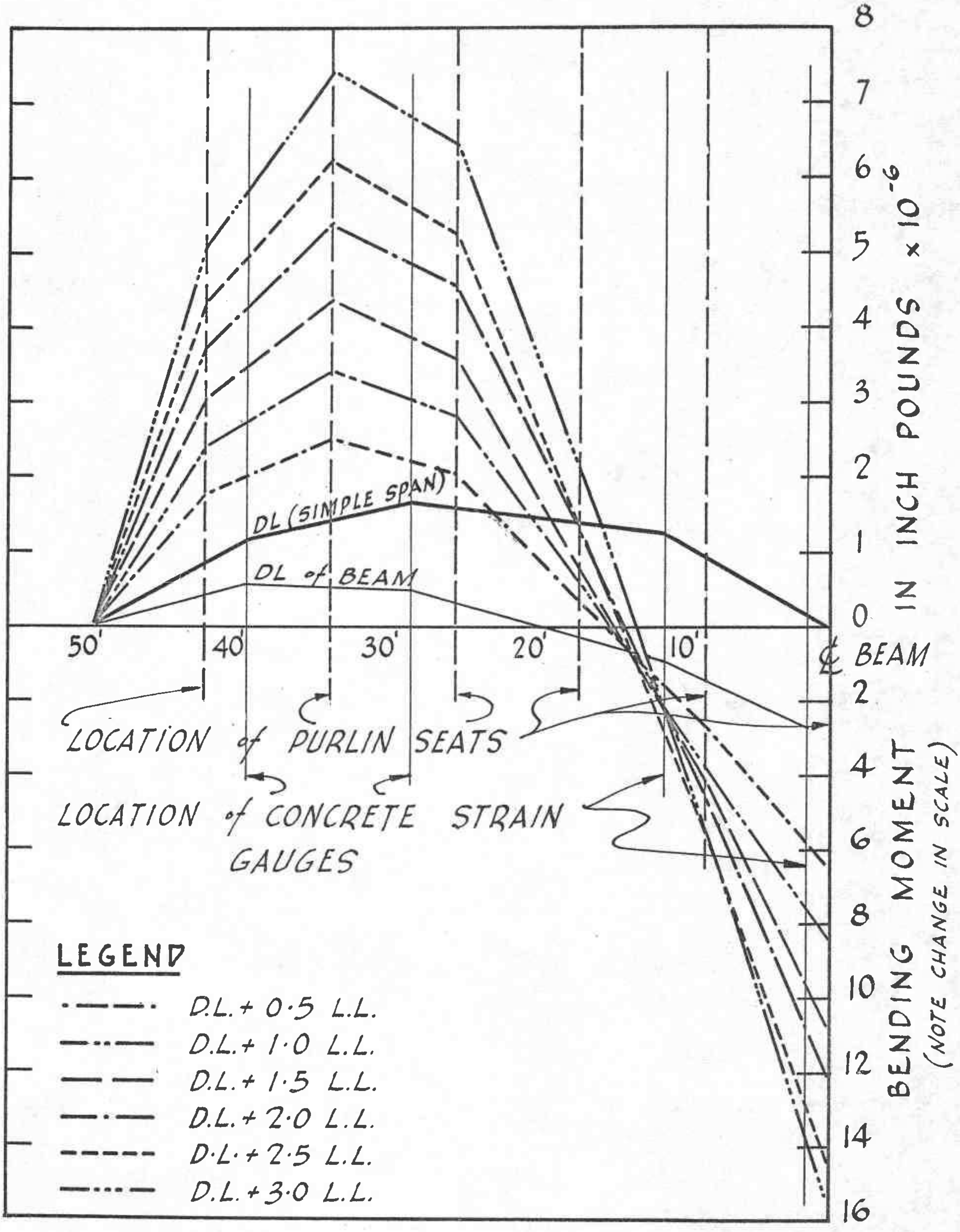


FIGURE 35

BENDING MOMENTS FOR VARIOUS LOADS

BR 3223 - 29

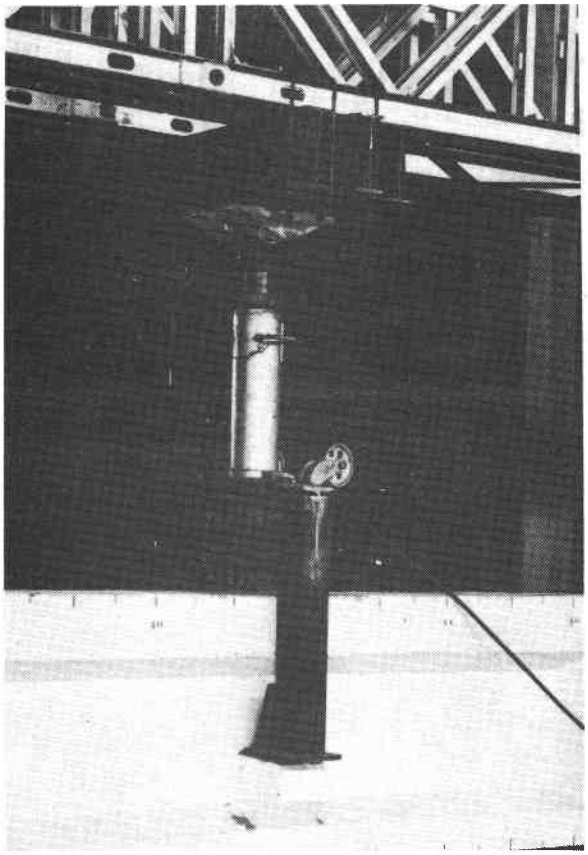


Figure 36 Test Set Up Used for Testing Purlin Seats.

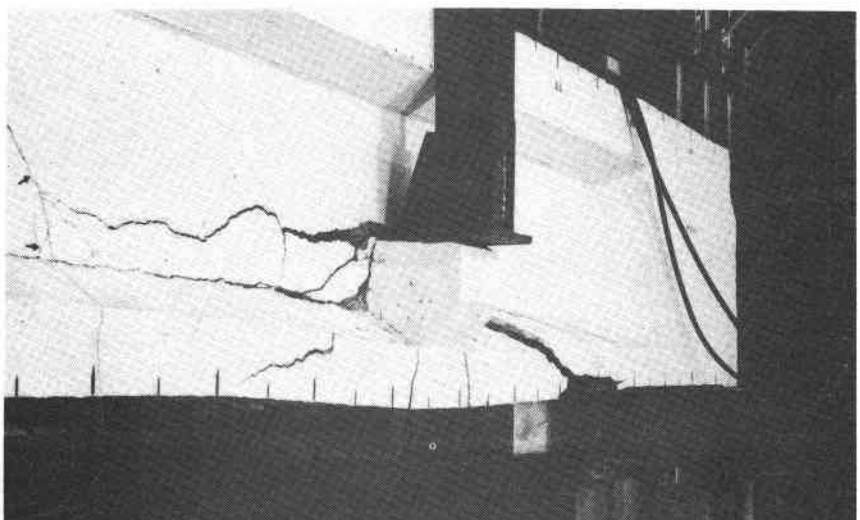


Figure 37 Cracking At Purlin Seat at DL + 13 LL.