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PRESTRESSED CONCRETE BRIDGE MEMBERS

Progress Report No.9

FIELD TESTS ON A PRESTRESSED CONCRETE MULTI-BEAM BRIDGE

by

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(Not for Publication)

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A B S T R A C T

This report describes a bridge test carried out by the concrete research group of Fritz Engineering Laboratory, Department of Civil Engineering and Mechanics at Lehigh University. It is part of a sponsored research program on prestressed concrete which has been carried on at the University since 1951.

The tested highway bridge with a clear span of 32 ft. and a width of 27 ft. is composed of 9 prefabricated, pretensioned concrete beams. Placed side by side, the beams are connected together by a steel bolt at midspan and have dry-packed continuous shear keys. The bridge was tested in the field under static and dynamic loading.

The purpose of these tests was to check the design criteria with special emphasis to determine the portion of the live load to be carried by each beam.

It was found that the bridge, designed under the assumption that each beam is to carry 30 percent of the right or left wheel loads of an H20-S16 truck, was very stiff. This is mainly due to an effective interaction of the beams which results in a bridge behavior approaching that of a plate.

The distribution of the live load was found to be very favorable. The largest static loading applied in the edge lane produced a bending moment of 148 percent of the design moment and caused a maximum deflection in the edge beam of 0.087 in., or 1/4690 of the span. For a truck in the center lane, the middle beam carried approximately 24 percent of the right or left wheel loads. For a truck in the edge lane, the edge beam carried approximately 46 percent of the right or left wheel loads.

The action of the bridge due to a truck moving at 25 mph over the bridge was satisfactory. In the first series of tests, where the truck travelled in the centerlane without any obstacles in its path, the maximum deflection was 114 percent of the corresponding static deflection. With the truck in the edge lane, the maximum deflection was 123 percent of the corresponding static deflection. In the second series of tests, where the truck travelled in the centerlane with a 2 in. plank in its path, the maximum deflection was 156 percent of the corresponding static deflection. With the truck in the edge lane and travelling over the 2 in. plank, the maximum deflection was 300 percent of the static deflection. It should be noted that the maximum deflection caused by dynamic loading in the edge lane was only 0.072 in.

It is hoped that the results of these tests can soon be substantiated by a forthcoming theoretical study after which definite recommendations about possible reductions of the design load of the beams could be made.

I. INTRODUCTION

1. Objectives:

The bridge test described in this report is a part of an extensive research program on structural members of prestressed concrete carried on at Lehigh University. The general purpose of this research program is to (1) check the validity of the design assumptions of structural members, (2) determine the effect of static and repeated loads and (3) to furnish data that may aid in the preparation of design specifications with the ultimate aim of contributing to the progress of prestressed concrete.

Two previous major reports[1,2]* presented the study of the endurance tests of two full-scale bridge members. The ever important problem of the bonding characteristics of prestressing strands has been and is still being investigated as part of the program. As a sequel to the study of the performance of such individual bridge members, the testing of bridges composed of such members is a logical extension of this research program.

In such bridges, the main problem to be investigated is the interaction of the beams and the determination of the portion of the total live load each beam must carry. This investigation has been divided into three phases; theoretical studies[3], field tests on actual bridges, and laboratory tests on a small bridge. The tests described in this report cover the first of the field tests planned for the near future.

2. Description of the Test Bridge:

The bridge for this first field test was selected in co-operation with Mr. L.A. Porter, Bridge Engineer of Pennsylvania,

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*Numbers refer to List of References.

and the late Mr. B.J. Baskin, Vice-President and Chief Engineer of the Concrete Products Company of America. It is located at the west end of Centerport in Berks County, Pa. on Route 06019, Station 354+58 and spans over the Centerport Creek. The bridge, which was erected in December, 1952, is shown in Figures 1,2,3. It has a clear span of 32 feet between the inside faces of the abutments and a nominal roadway width of 25 ft.-4 in. Two 8 in. curbs are provided on each side. A two-inch layer of bituminous material forms the wearing surface. The bridge is composed of 9 prefabricated, pretensioned beams of the type previously tested at Lehigh University and described in Progress Report No.5[1]. Each beam is 36 in. wide, 21 in. deep, and has an overall length of 35 ft.-6 in. (Fig.4). The cross-section is rectangular except for the keyways on the sides of the beams near the top and the two hollow circular cores 12 1/2 in.dia. in the center of the cross-section. Used to reduce the dead weight, these hollow cores extend the total length of the beam except for a 2 ft. solid section at each end and a 10 in. solid section at the center of the span where the lateral tie rod goes through the beam. The 8 in. curb was poured after the release of the prestress in the edge beam proper and anchored to it by means of hooked dowels placed for this purpose.

The bridge was erected by placing the beams side by side on the abutments using a large truck crane. The beams had two 2 in. diameter holes cast in each end and were made to line up with corresponding holes in the abutments. Finally, anchor bolts of 3/4 in. diameter and 26 in. length were inserted and grouted to tie down all beams to the abutments. Steel guard rails are bolted to the outside of the curb beams.

The mortar used for dry packing is composed of two portions of sand, one portion of cement, and one portion of "Groutex". This latter material is added to the mortar to minimize its shrinkage thus preventing cracking between the old concrete of the beams and the mortar of the shear keys. The amount of water generally added is enough to make the mortar stick together when pressed between the palms of the hands. The mixture thus produced is then rammed into the shear keys with tamping rods.

3. Design of the Beams:

The beams were designed according to the AASHO 1949 Specifications and the "Specifications for Prestressed Precast Reinforced Concrete Bridge Deck" dated May 1, 1952 and prepared by the Pennsylvania Department of Highways. The design live load was a H20-S16-44 standard truck, having a minimum rear axle spacing of 14 ft. The values for static loading were increased by 30 percent to take the effect of impact into consideration. It was assumed that each beam carried 4/5 of the wheel load with the remaining 1/5 distributed among the adjacent beams. Each beam was prestressed by 56 high-tensile wire strands with a nominal diameter of 1/4 in. The total area of the strands was 1.97 square inches and their location is shown in Fig.4. Initially the strands were prestressed to 135,000 psi. A loss of 20 percent of the steel prestress was assumed for shrinkage, plastic flow, elastic shortening and creep. Four No.5 reinforcing rods were placed longitudinally near the top of each beam and supported vertical stirrups and a wire mesh.

The contractor reported that a cylinder strength of 3300 psi at release of the prestress and a minimum of 5000 psi at 28 days was required. There are no records giving this data.

II. TEST PROGRAM AND PROCEDURE

1. Test Program:

The test program included principally static and dynamic tests.

(A) Static Tests

The purpose of the static tests was to determine the percentage of live load carried by each beam of the bridge. This was accomplished by measuring the deformed shape of the bridge loaded with two different types of trucks to be described later. The loading of the bridge was performed by positioning these trucks successively at the quarter-point and midspan in both the edge and center traffic lanes. These positions are given in the several figures showing the deflection surfaces. (Figs.14 to 25).

(B) Dynamic Tests

These tests were intended to provide some information regarding the general behavior of the bridge under dynamic loading. No attempt was made to fully study the lateral load distribution under this type of loading. The deflection at a few critical points on the bridge were observed while a truck was driven over it at a speed of 25 mph. A two-inch plank was later placed across the bridge at different points along the path of the truck.

2. Test Procedure:

The field work was carried out during the week of July 19, 1954. The instrumentation was installed in two days and after completion of the test was dismantled in half a day. On July 21, 1954 preliminary tests were made to check the proper adjustment of the instruments and to coordinate the efforts of the testing team. The main tests were performed on July 22, 1954 and were witnessed by several committee members and others listed at the end of this report.

(A) Instrumentation

The instrumentation used in this bridge test is shown in Fig.5 and consisted of Ames dials, level bars, a Whittemore gage, and electronic equipment to record the deflections under dynamic loading.

(a) Deflection gages:

39 Ames deflection gages, least count ranging from 1/1000 in. to 1/10,000 in., were located as shown in Fig.5. These gages were placed in a manner that emphasis was given to the deflection of each beam at midspan and to the accurate determination of the deformed shape of the north-west quadrant of the bridge. The rest of the gages were used to check the symmetrical behavior of the bridge. A simple timber scaffolding as shown in Fig.6 was erected to provide the supporting structure for the gages and was completely independent of the bridge.

(b) Level bars:

The transverse rotation of three different beams at midspan was measured by means of level bars. A level bar consists of a 20-in. aluminum bar having a sensitive level-bubble rigidly attached and is provided with a simple pin support at one end, and a micrometer screw-support at the other. The bar was simply fastened to a beam by means of the end fittings and was leveled by centering the bubble with the micrometer screw support. When the live load was applied, the beam rotated, and the level bar rotated with it. The angle of rotation would then be obtained from an Ames Dial mounted on the level bar at the adjustable end. The Ames Dial measured the amount of movement in the vertical plane at the end of the bar needed to recenter the level bubble. The angle of rotation, of course, was directly proportional to this measurement.

(c) Whittemore gage:

Strain readings were taken at midspan of several beams. For this purpose steel plugs to be used with a 10-in. Whittemore gage were glued to the underside of the beams with sealing wax. Due principally to the small change in readings upon application of the load and the large influence of the temperature, these strains were too inconsistent to be of any use.

(d) Dynamic deflection gages:

One of the three dynamic deflection gages used in these field tests is shown in Fig.7. It consists of a small cantilever beam fixed to a base plate which was mounted on the timber scaffolding. A short vertical rod was connected to the free end of the cantilever and made contact with the concrete beam in such a way that the cantilever deflected with the beam. Two SR-4 strain gages mounted on the top and the bottom of the cantilever provided the means of measuring its deformation which was in proportion to the deflection of the beam determined by laboratory calibration. A two-channel Brush-recorder registered the readings. (Fig.8).

(B) Loading

Two trucks shown in Figs. 9,10,11, and 12 were used to apply the live loads on the bridge. Fig.13 shows how the test loading compared with the AASHO loading on which the design of the bridge was based.

(a) Scale truck:

Through the cooperation of the Bureau of Standard Weights & Measures, Commonwealth of Pennsylvania, a special truck used to calibrate large platform scales was made available for the test. The total weight of this 3-axle truck is 37,900 lbs. including 17,250 lbs. made up of removable weights. The wheel locations of the truck are shown in Fig.12.

(b) Tractor-trailer truck:

The Concrete Products Company of America supplied a tractor-trailer truck which was loaded with four precast bridge members similar to those used in the test bridge. The axle load of this loaded truck was 33,700 lbs. as measured on a field scale and was the loading used for the dynamic tests. Additional weights from the scale truck were added over the rear axle of the trailer to produce an axle load of 47,700 lbs. for the static tests. The axle dimensions and weights are shown in Fig.12.

In the static tests the axle loading for the centerlane was concentrated on the middle beam by using a pair of jacks. This method is shown in Fig.11.

III. TEST RESULTS

1. Static Tests:

(a) Deflections

As the bridge tests were carried out on warm summer days, the variation in temperature during the day affected the readings extensively. To minimize these effects, frequent zero readings were taken with no load on the bridge and the intermediate readings were corrected, assuming a linear variation of the temperature change between two consecutive zero readings. However, this procedure does not take into account any possible residual deformation of the bridge, such as the plastic deformation of the bridge, the plastic flow of the concrete, or a possible residual relative displacement of adjacent beams after tests. Since the bridge had been in service for a year and a half, the applied short time test loading which was insufficient to crack the beams, should have produced either none or only minor plastic deformations of the concrete. Reliable measurement of any residual relative displacements of adjacent beams would have been possible only by placing additional deflection gages of very high sensitivity across the joints of two adjacent beams. A much larger number of very sensitive gages would have been necessary in this first bridge test. The relative deflection of the component beams will be extensively investigated on the short-span bridge to be assembled in the laboratory.

It was stated earlier that vertical deflections were measured at close intervals in the northwest quadrant of the bridge and that control gages were placed only at critical points in the other quadrants. By placing the truck loads at points symmetrical to each other with respect to the bridge axis and utilizing the relation existing between symmetrical and reciprocal deflections, the deformed shape of the complete bridge deck was obtained. The symmetrical deflections were checked with the control gages, and were found to be close to the values of their symmetrically opposite points. To reduce the magnitude of the deviation between readings, average deflections were calculated for the different symmetrical loading positions. The resulting deflections are plotted in the figures listed below which show the deformed shape of the bridge in isometric views. Positive deflections are plotted upwards and their magnitudes are shown at the gage points in thousands of an inch. In the right-hand corner a plan view indicates the type and the location of the loading. The deformed shape of the bridge deck is for the following:

Fig.14 with the scale truck in the centerlane, and the resultant of the rear axles at the quarter point.

Fig.15 with the scale truck in the centerlane, and the resultant of the rear axles at midspan.

Fig.16 with the scale truck in centerlane, and the resultant of the rear axles at the three quarter points. The front axle is off the bridge.

Fig.17 with the scale truck in the edge lane, and the resultant of the rear axles at the quarter points.

Fig.18 with the scale truck in the edge lane, and the resultant of the rear axles at midspan.

Fig.19 with the scale truck in the edge lane, and the resultant of the rear axle at the three quarter points. The front axle is off the bridge.

Fig.20 with the rear axle of the tractor-trailer truck jacked up at the quarter point in the centerlane concentrating the axle load on the center beam.

Fig.21 with the rear axle of the tractor-trailer truck jacked up at midspan in the centerlane, concentrating the axle load on the center beam.

Fig.22 with the rear axle of the tractor-trailer truck jacked up at the quarter point in the edge lane.

Fig.23 with the rear axle of the tractor-trailer truck jacked up at midspan of the edge lane.

Fig.24 with the tractor-trailer truck and the scale truck back to back in the centerlane at midspan.

Fig.25 with the tractor-trailer truck and the scale truck back to back in the edge lane at midspan.

From a study of the above figures one notices that the largest measured deflection is 0.087 in. which was observed at midspan of the upstream edge beam under a loading of the tractor-trailer truck and the scale truck placed back to back at midspan of the edge lane. (Fig.25). This deflection produced by a bending moment of 148 percent of the design moment (without impact) represents 1/4690th of the bridge span.

A deflection of 0.071 in. was observed in the middle beam when the rear axle of the tractor-trailer truck was placed at midspan and jacked up over the centerlane. (Fig.21). This deflection caused by a bending moment of 118 percent of the design moment without impact is 1/5710th of the bridge span. These deflection-span ratios clearly indicate that the bridge is very stiff.

<u>(b) Level bar readings:</u>

The deflection ordinates on the above listed figures also show a relative deformation of adjacent beams giving a dishing effect to the bridge surface. This distortion may be the result of either of the following two conditions or a combination:

The first condition exists when two adjacent beams undergo a slight vertical movement relative to each other with their abutting faces simultaneously going through a partial but identical rotation. Thus the deflection curve in the lateral direction of the bridge would have steps at each beam face. Shear is transferred across the joint by friction or forces of similar nature. This shear would be different on the opposite sides of a beam producing a torsional moment which in turn tends to twist the beam. The accuracy of this can be confirmed by applying the conditions of equilibrium which must exist between the beams.

The second condition occurs when no relative movements between the adjacent beams exists. Since the centers of the beam-sections deflect different amounts, twisting of the beams must necessarily also accompany the deformation of the beams to satisfy not only the equilibrium but also the conditions imposed by the deformations. In this case the deflection curve in the lateral direction would be continuous. Therefore, when the deformation is dependent, to the extent described above, on the torsional rotation of each beam, the behavior of such a bridge can be considered to be similar to that of a plate.

To investigate the degree to which these conditions existed in the test bridge, three level bars as described in Section II were attached transversely to the beams at midspan. The measured lateral rotations and the corresponding vertical deflections of Figs. 21 & 23 together with the loading positions are shown in

Fig.26. From these figures it can be seen that the measured rotations of the beams closely agree with the slope of the transverse measured deflection curves. The small deviations at some of the points lie within the range of accuracy of the measuring instruments. However, one cannot completely exclude the possibility of a slight relative movement between beams, especially near the load points. Nevertheless it is apparent that in general the elastic curve in the lateral direction of the bridge conforms with the second condition described above. It is therefore reasonable and justifiable to assume in any theoretical investigation a continuous lateral deflection curve, hence excluding any relative movement between abutting faces in the vertical plane.

(c) Cracking:

Inspection by naked eye of the bridge prior to the test revealed the absence of shrinkage or any other cracks. To aid in detecting the formation of cracks as a result of superimposed loads the underside of the test bridge was white-washed. However, no cracking was noticed under the action of any of the applied loads. The type of construction of the bridge did not permit the inspection of the shear keys to determine if they were affected.

(d) Lateral load distribution:

The design specifications now used by the Pennsylvania Highway Department assumes 80 percent of the wheel load of an H20-S16 truck is taken by each beam. These assumptions are based on an interpretation of design section 3.3.1b of the 1949 AASHO Specifications. This could also be interpreted to mean that the wheels of one side of the truck are positioned over a single beam which is then assumed to carry 80 percent of these wheel loads or 40 percent of the total truck load.

A more accurate analysis which will be given in Progress Report No.10 assumes the bridge to be acting as a homogenous plate with different bending stiffnesses in the longitudinal and lateral directions. However, computations in this analysis are quite extensive. The design procedure for bridges can be greatly shortened if coefficients of the lateral load distribution are computed from the general plate theory for different sizes of beams and different bridge spans and widths. The bridges can then be designed with the conventional beam theory, considering the beams as simply supported and loaded with the portion of the total live load determined by using the coefficients mentioned above.

In Progress Report No.10 it will be shown that the coefficient of lateral load distribution S_i can be defined as the ratio of the bending moment M_x per unit width of the beam i to the average bending moment M_x average. The latter is the bending moment per unit width in the cross-section under consideration and produced by a particular loading position, assuming that all the beams of the bridge carry the same portion of the load; or

$$M_x \text{ average} = \frac{1}{2b} \int_{-b}^{+b} M_x \, dy \sim \frac{1}{2b} \sum_{i=1}^{n} M_{xi} \cdot a \qquad (1)$$

where 2b is the width of the bridge, a the width of an individual beam, and n the number of the beams. It is evident that this value of M_x average is identical with the total external live load moment divided by the width of the bridge. S_i can then be expressed as:

$$S_i = \frac{M_{xi}}{M_x \text{ average}} \qquad (2)$$

This coefficient is very suitable for a non-dimensional representation of the lateral load distribution in form of graphs and tables since it is independent of the number of beams incorporated in a bridge. However, for a direct comparison with the present design procedure, the lateral load distribution coefficients in Report 10 will be given in percent of the applied live load carried by each beam. These values can be obtained from equation (12) performing the following modification:

$$S_i^* = \frac{S_i}{n} \cdot 100 \text{ (in percent)} \qquad (3)$$

The same coefficients as defined in equation (2) are used in several theoretical studies of similar bridge systems, published in the European literature[4,5,6]. However, these coefficients are determined only for a live load of sinusoidal shape, the equation of which is $p=p_o \sin\frac{x}{l}$, where p_o is the maximum load intensity at a distance x from one of the supports. The authors of these European papers consider this loading as a sufficiently close approximation for any loads to be encountered in the design of bridges.

The assumption of a sinusoidal type of loading has the further advantage that the coefficients of the lateral load distribution as defined above are also equal to the ratio of the deflection of the beam i to the average deflections of all the beams in a cross-section. For other types of loading, such as an axle load, the two ratios are not equal and the coefficients obtained using the deflections are only a first approximation for the coefficients of the lateral load distribution. The correct lateral load distribution can therefore be obtained only by considering the bending moments produced in each beam.

Theoretically it is possible to determine from the measured deflections the curvatures of the deformed shape of the bridge deck and thus the bending moments. However, the resulting values include in general rather large errors, since they are obtained by differentiating twice an approximate curve drawn through 3 experimentally determined points, and the moment is not the maximum but the average over the segments. (Method of Finite Differences). In the test bridge the gage points were relatively far apart and as it would be expected the bending moments, so determined, were consequently inconsistent. For this reason, the lateral load distribution coefficients for the test bridge were based on the deflection ratios.

The exact values will be computed in Progress Report No.10 and the amount of error involved in the above approximation will be discussed.

Fig.27 shows the lateral load distribution curve obtained from the deflections for any longitudinal truck position when in the centerlane. The cross-section of the bridge and the lateral loading position are schematically indicated at the top of the graph. The position of the gage lines are marked and numbered from 1 to 9 on the abscissa. The ordinate scale denotes the percentage of the total applied load that is carried by each beam. The three curves designated by solid lines give values obtained from deflection measurements involving only the scale truck and the combination of both trucks in the centerlane. The upper and lower curves designated by solid lines give maximum and minimum values observed in any of the gage lines and/or any longitudinal loading position in the centerlane. The broken line that is connected to the maximum value curve connects points obtained by concentrating the applied load on the middle beam by jacking up the rear axle of the tractor-trailer truck. The heavy solid line represents the average values obtained from scale truck and combination of both trucks; and the broken line connected to this curve represents average values taking into account the effect of concentrated load from the jacks applied to the tractor-trailer truck.

Considering the middle beam which carries the largest percentage of the load, it can be seen that the load distribution factor is 16.0 percent on the average and 16.9 percent as a maximum value. The latter increases to 20.5 percent for the axle load concentrated over the beam.

Fig.28 is the complement of Fig.27 and shows the distribution factors for loading positions in the edge lane. As can be expected the edge beam carries, in the case of wheel loadings, the largest percentage of load; namely 20.6 percent on the average and 22.6 percent as a maximum. For the axle loading jacked up and concentrated over the second and third beam, the portion of the load supported by the second beam reaches a maximum value of 23.9 percent.

However, mention should be made that for loads applied at the quarter points the observed load distribution is slightly less effective than for loading positions at midspan. These deviations are shown on the maximum and minimum value curves. The above mentioned distribution ratios have to be compared with the design provisions of 40 percent of the axle loads. However, one should be aware that the coefficients shown in Figs.27 are based only on the relative deflections of the beams which are reasonable approximations and as mentioned before the exact values will be given in Progress Report No.10.

(E) Modulus of Elasticity

An approximate value of the modulus of elasticity E for the bridge was obtained in a way similar to the method used to determine the coefficients of the lateral load distribution. This was done by comparing the sum of the measured deflections in all the beams in a cross-section of the bridge with the theoretical deflection of one beam, loaded with the total applied live load. The theoretical deflection, a function of the modulus of elasticity, was computed assuming an uncracked section. Neglecting the deformations due to the bending in the lateral direction of the bridge and the twisting of the beams, the above mentioned deflections are identical and this relationship was used in the determination of the modulus of elasticity. This was done for 12 different loading positions and for all the gage lines in the lateral direction in which complete deflection readings were available, resulting in 75 values for the modulus of elasticity. The average value for E was 6.68×10^6 psi with a maximum of 7.73×10^6 psi and a minimum of 5.38×10^6 psi. These values and the variation may seem high but one must remember that the bridge was made out of high quality concrete and had been in service for a year and a half at the time of the test. Furthermore, the values were obtained by neglecting the above mentioned portion of the deformation.

2. Dynamic Tests:

Two sets of dynamic tests were run, namely: (a) with the truck travelling at 25 mph over the bridge, and (b) with the truck travelling 25 mph and striking a single 2 in. plank. Typical time-deflection graphs from these tests for the tractor-trailer truck travelling toward Centerport are given in Figs.29

and 30 for truck in centerlane and downstream edge lane, respectively. Fig.31 shows the relationship between truck position and beam deflection for the second set of tests.

(a) Truck moving unobstructed at 25 mph over the bridge:

The upper graph in Fig.29 shows the time deflection curve recorded at midspan of the middle beam for the truck moving in the centerlane of the bridge. A maximum deflection of 0.041 in. was determined when the trailer axle was at midspan. The corresponding static deflection for the same loading measured with the electric deflection gage was 0.036 in. By comparing these two values it is evident that the dynamic deflection was 13.9 percent larger than the one caused by static load of the truck. If the impact effect is defined as the increase in deflection due to a rapidly applied compared with a gradually applied loading, it is in this case 13.9 percent.

Fig. 30(a) shows the time deflection curve recorded at midspan of the downstream edge beam for the truck travelling in the downstream edge lane, the outside tires about 1 1/2 ft. from the curb. The maximum dynamic deflection recorded was 0.032 in. and the equivalent static deflection 0.026 in. resulting in an impact fraction of 23 percent. These impact fractions can be compared with the one prescribed by the 1953 AASHO Specification 3.2.12.c found as follows:

$$I = \frac{50}{34+125} = 31.4 > 30 \text{ percent maximum}$$

Study of the overall shape of the time deflection curve, shows that only minor vibrations occurred as the truck travelled across the bridge. This is probably due to the smooth bituminous wearing surface and the relatively large stiffness of the bridge.

(b) Impact tests:

The impact tests were performed by driving the truck on the bridge at 25 mph and over a plank of dimensions 2 in. by 10 in. placed flat across its path at midspan. Fig. 29(b) shows the time deflection curve measured at midspan of the center beam for impact produced in the centerlane. Fig. 30(b) shows the time-deflection curve measured at midspan of the downstream edge beams for impact produced in the edge lane. These curves were most useful in determining the natural frequency of the bridge as a result of impact loading. The frequency was found to be approximately 10 cycles per second, which agrees very closely with the theoretical natural frequency of 9.5 cycles a second computed for one beam using a modulus of elasticity of 6.68×10^6 psi.

Although the impact produced by a truck running over a two-inch thick plank is much higher than the impact under normal traffic conditions, it may be approached when only one or two emergency chains are used on the tire of a fully loaded truck. It is also of interest to compare the dynamic deflections with the corresponding static deflections. However, since the vibrations are complicated and a more extended analysis would be outside the scope of this report such a comparison will have to be restricted to a few points. In fact it can be seen that consistently in all the impact tests performed on this bridge the less loaded rear axle of the tractor produced the largest deflections whereas the deflections resulting from the impact of the heavier trailer axle were smaller. (See Fig.31) No attempt is made to interpret this observation, but should one wish to further analyze this behavior it should be kept in mind, that the mass of the bridge is about 3 1/2 times larger than the total mass of the truck. Furthermore, the tractor-trailer with its springs and shock absorbers forms a rather complicated dynamic system.

Due to the inevitable variations in the speed and the path of the truck in the different runs, an accurate superposition of corresponding points in the two graphs of Fig.29 and Fig.30 is not possible. The corresponding static deflections in the lower graphs were therefore approximately determined as the midpoint of consecutive minimum and maximum values. The increase in deflections due to the impact was then measured as shown in the figures and the impact fractions computed. For the truck in the centerlane and the impact fraction corresponding to the rear axle of the tractor was determined to be approximately 106 percent whereas for the trailer axle it was approximately 44 percent. The corresponding impact fraction values for the truck in the downstream edge lane were approximately 200 percent and 56 percent respectively. Recalling that a sudden application of the load without impact increases the static deflection up to 100 percent, the observed increase of the deflections due to the impact loading can be considered as being reasonable.

IV. CONCLUSIONS

A 27-ft. wide highway bridge with a roadway width of 25 ft. 4 in. and a clear span of 32 ft. (Figs.1,2,3) composed of 9 prefabricated, pretensioned concrete beams placed side by side, connected by a single steel bolt at midspan and by dry packed continuous shear keys, was tested in the field under static and limited dynamic loading. The results of these tests are summarized below:

General:

1. The bridge proved to be entirely structurally satisfactory for its intended purpose.
2. Although the bridge had been in service for over a year and a half, no cracks due to shrinkage, or other causes, or due to the application of the test loads could be detected on the beams.
3. The largest static loading applied in the edge lane, produced a bending moment without impact of 148 percent of the design moment and caused a maximum deflection in the edge beam of 0.087 in. or 1/4690th. of the span.
4. The action of the component beams, as determined through their recorded deflections and rotations in the transverse plane, supports the conclusion that the overall behavior of the bridge approaches that of a homogeneous plate. This interaction was mainly due to the shear keys and the single transverse bolt. However, it was not irrefutably proved that the shear keys prevented any relative movement of the contact faces of any adjacent beams. A careful investigation

on a small span bridge and possibly on a special test specimen could yield a more definite answer.

5. The field tests supports the assumption for a theoretical investigation that the deflection in the lateral direction is of a continuous character.

6. A modulus of elasticity for the bridge of 5.38×10^6 to 7.73×10^6 with an average of 6.68×10^6 psi was determined from a comparison of the measured with the theoretical deflection.

Lateral Load Distribution and Design of the Bridge:

7. The test results permitted an approximate determination of the lateral load distribution as follows:

8. For a truck in the centerlane the middle beam carried approximately 17 percent of the total axle loads on the bridge or 34 percent of its left or right wheel loads. See Fig.27. When an axle load of 47.7 kips was concentrated on the middle beam at midspan through the use of jacks, the middle beam carried 20.5 percent of this load.

9. For a truck in the edge lane, the edge beam carried approximately 23 percent of the total axle loads on the bridge or 46 percent of the left or right wheel loads. (Fig.28).

10. The present design assumption, which assigns 40 percent of the standard truck or 80 percent of the left or right wheel loads seems to be conservative for static loading.

11. Recommendations concerning a possible reduction of

the portion of the live load that the engineer must design for, cannot be based on these tests alone. Adequate additional information should be gained from the theoretical study now in progress as well as the laboratory bridge testing that is contemplated for next year.

Dynamic Tests:

12. The dynamic tests also indicated a considerable stiffness of the bridge.

13. A truck travelling at 25 mph over the bridge without any obstacle in its path produced only slight vibrations. As a result, with the truck in the centerlane the static deflection of the beams in that lane is increased by 13.9 percent. With the truck in the edge lane the deflection in the edge beam is increased by 23 percent. Figs.29 and 30. These increased deflections mentioned are within the prescribed 30 percent maximum allowance in the AASHO Specifications.

14. A truck running over a 2-in. thick plank at 25 mph caused deflection increases ranging between 56 percent and 200 percent of the corresponding static deflections. It is not certain how these percentages should be compared with the prescribed 30 percent allowance of the AASHO, because of the severity of the impact.

15. The natural frequency of the bridge was found to be about 10 cycles a second, which checks well with the theoretical value of 9.5 cycles a second for a single beam.

V. ACKNOWLEDGEMENTS

This bridge test is a part of an investigation, carried out under the auspices of the Lehigh University Institue of Research using the facilities at Fritz Laboratory and under the guidance of the Lehigh Prestressed Concrete Committee of which Mr. A.E. Cummings is Chairman. This committee, representing the sponsoring organizations, is composed of the following members to each of whom the authors wish to express their gratitude for their technical and financial contribution.

Mr. A.E. Cummings, Reinforced Concrete Research Council
Mr. L.A. Porter, Pennsylvania Department of Highways
Mr. J.L. Stinson, U.S. Bureau of Public Roads
Mr. Neil VanEenam, U.S. Bureau of Public Roads
Mr. B.J. Baskin, Concrete Products Co. of America (deceased)
Mr. F.S. Burtch, John A. Roebling's Sons Corporation
Mr. H.K. Preston, John A. Roebling's Sons Corporation
Mr. W.O. Everling, American Steel and Wire Co.

This research program on prestressed concrete bridge members was, since its inception and until this phase, under the co-directorship of Prof. W.J. Eney, Head of the Department of Civil Engineering and Mechanics and Director of Fritz Engineering Laboratory. The other co-directors have successively been Dr. A.C. Loewer, Jr., Dr. K.E. Knudsen, Prof. M.J. McCrodden and Dr. C.E. Ekberg, Jr. With this phase, Dr. C.E. Ekberg, Jr. assumed the full directorship with Prof. A. Smislova serving as assistant project director, Prof. W.J. Eney as a consultant and editor and A. Roesli directly in charge of the test. Mr. Roesli planned the testing according to the many bridge tests carried out by the Swiss Federal Research Laboratory (EMPA)[7] in some of which he was privileged to take an active part.

Special mention is due Mr. L.A. Porter, State Bridge Engineer and Mr. E.J. Ferer, Bridge Engineer of District 5 Pennsylvania State Highway Department for the generous help in installing the test set-up. The Concrete Products Company of America, furnished the tractor-trailer truck and also helped in installing the test set-up. The scale truck was loaned for the duration of the tests by the Pennsylvania Department of Internal Affairs, Bureau of Weights and Measures.

Assistance during the testing period was given by members of the research staff of Fritz Engineering Laboratory as mentioned in the list of observers of the test, and by Mr. I.J. Taylor who was in charge of the instrumentation for the dynamic tests.

The large amount of work involved in analyzing the test data was done with the help of Messrs. A.N. Sherbourne, M.J. Giraldi, L.J. Debly and I.M. Scott, draftsman. Mrs. V.H. Olanovich typed the report.

To all those mentioned above and the Fritz Engineering Laboratory mechanics and technicians under Mr. K.R. Harpel, laboratory foreman, the authors are grateful.

VI. LIST OF OBSERVERS AND PARTICIPANTS IN THE BRIDGE TEST

Mr. L.A. Porter	Pennsylvania Department of Highways
Mr. E.J. Ferer	Pennsylvania Department of Highways
Mr. R.C. Updegraff	U.S. Bureau of Public Roads
Mr. B.J. Baskin	Concrete Products Company of America
Mr. S.L. Selvaggio	Concrete Products Company of America
Mr. G. Spare	American Steel & Wire Div., U.S. Steel
Mr. F.S. Burtch	John A. Roebling's Sons Corporation
Mr. L.E. Hill	John A. Roebling's Sons Corporation
Mr. Wm. Tilton	Lanar Pipe & Tile Co., Grand Rapids, Mich.
Prof. W.J. Eney	Lehigh University
Dr. C.E. Ekberg, Jr.	Lehigh University
Prof. E.K. Muhlhausen	Lehigh University
Mr. I.J. Taylor	Lehigh University
Mr. Y. Fujita	Lehigh University
Mr. M.J. Giraldi	Lehigh University
Mr. A.N. Sherbourne	Lehigh University
Mr. M.W. White	Lehigh University
Mr. A. Roesli	Lehigh University
Prof. A. Smislova	Lehigh University

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May, 1954

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LATERAL LOAD DISTRIBUTION IN MULTI-BEAM BRIDGES
(In Preparation)

4. Y. Guyon
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September-October, 1946, pp. 553 to 612

5. Ch. Massonnet
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METHOD OF CALCULATION FOR BRIDGES WITH SEVERAL LONGITUDINAL BEAMS, TAKING INTO CONSIDERATION THEIR TORSIONAL RESISTANCE
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6. P.B. Morice and G. Little
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Structural Engineer, March 1954

7. "Versuche und Ertahrungen an Eisenbeton-Bauwerken in der Schweiz"
TEST RESULTS AND EXPERIENCE WITH REINFORCED CONCRETE STRUCTURES IN SWITZERLAND
Swiss Federal Research Laboratory (EMPA) Report No.99

Fig. 1 General View of the Test Bridge

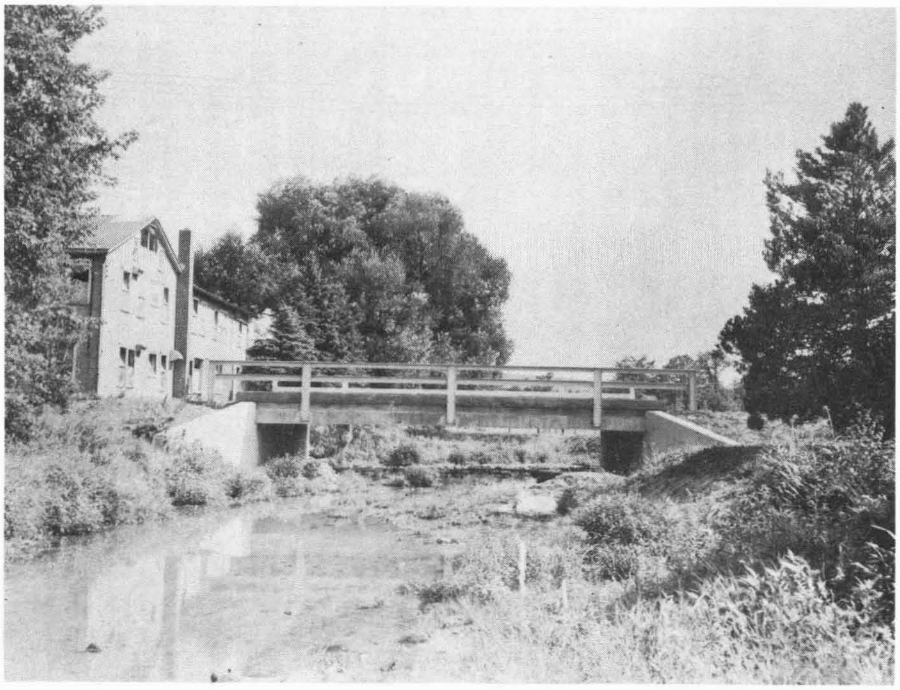


Fig. 2 Side View of the Bridge

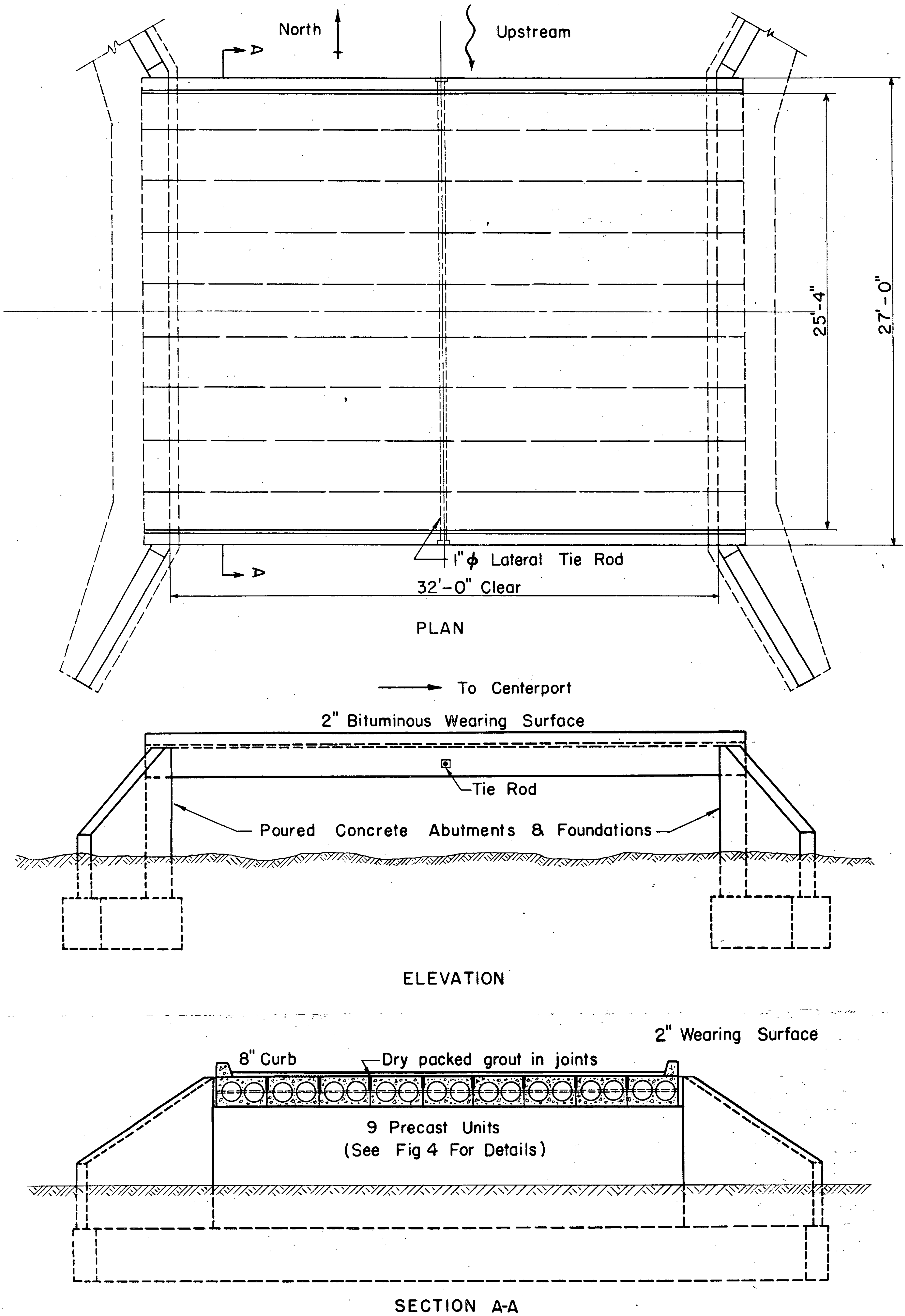
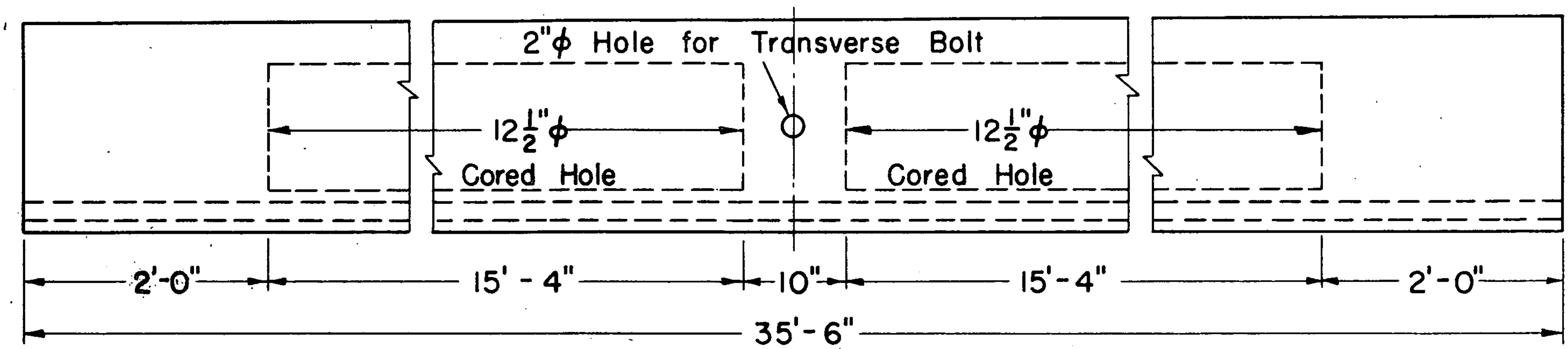
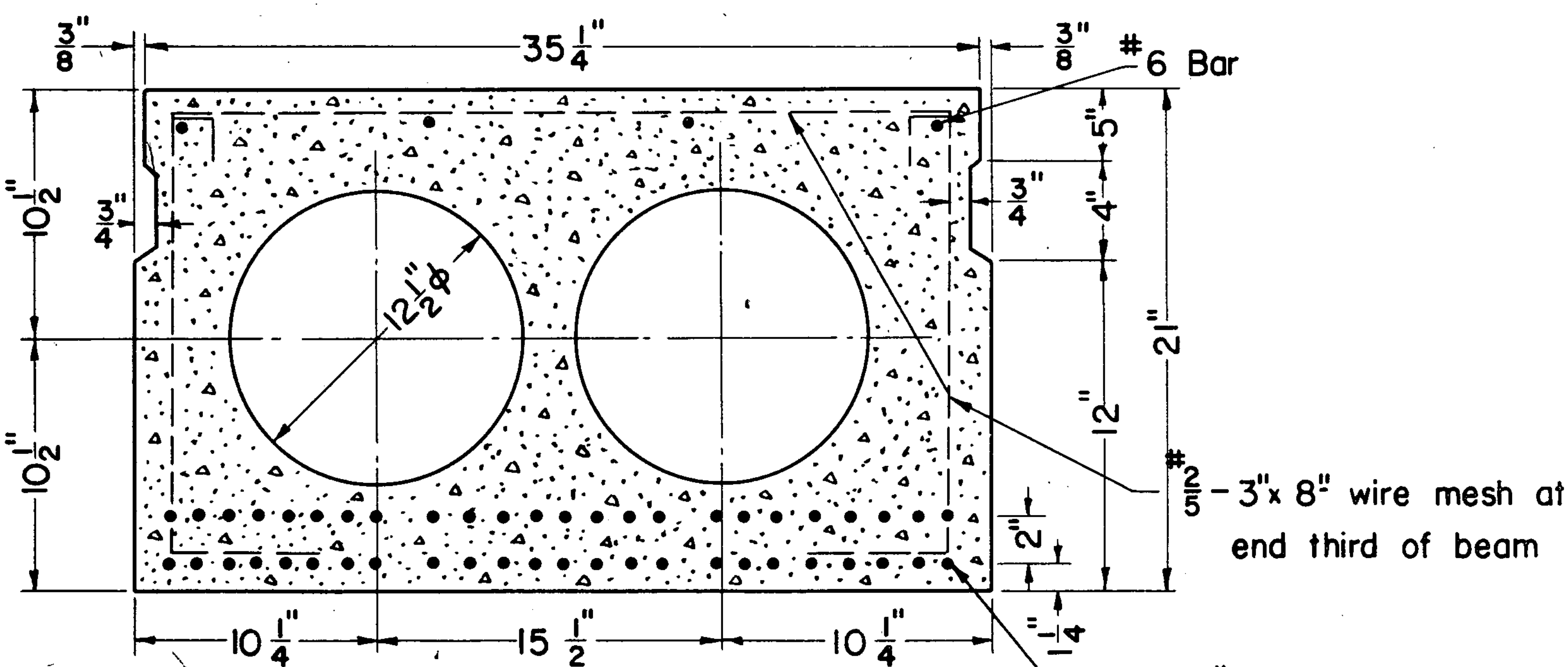


Fig. 3 General Arrangement of Bridge (Scale - 3/16" = 1'-0")



(a) Side View
Scale: 3/4" = 1'-0"



(b) Cross Section
Scale: 1/8" = 1"

FIG. 4 DETAILS OF BEAMS

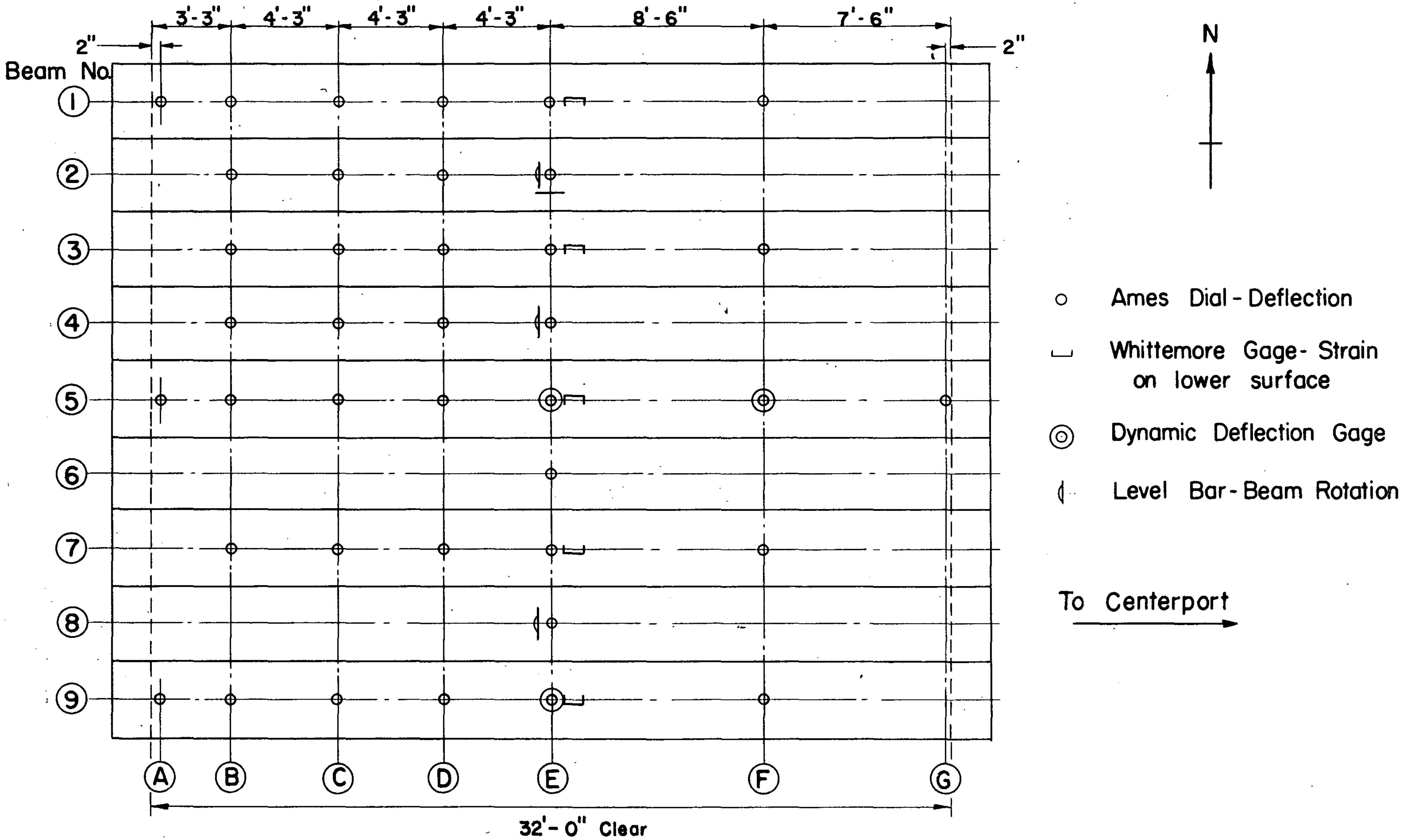


FIG. 5 INSTRUMENTATION

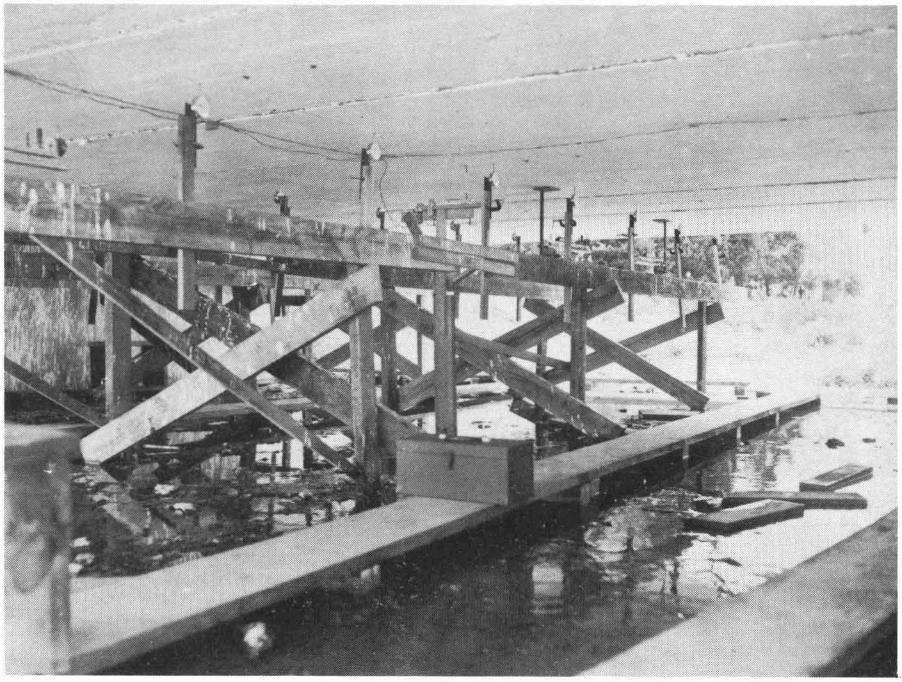


Fig. 6 General View of the Test Installation

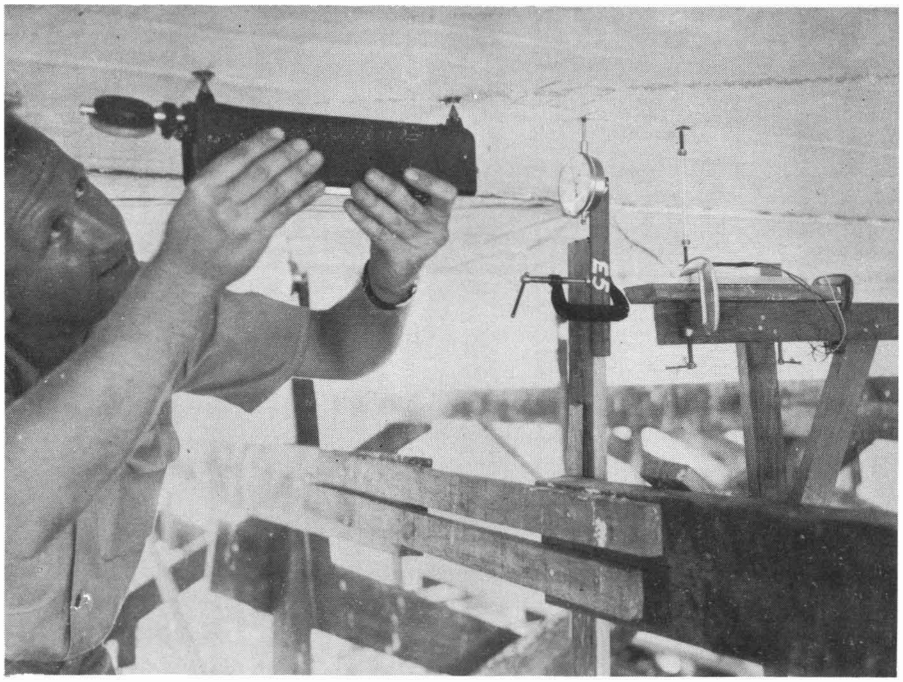


Fig. 7 Detail of the Installation from Left to Right
(a) Whittemore gage for strain measurements
(b) Ames gage for the deflection
(c) Dynamic gage

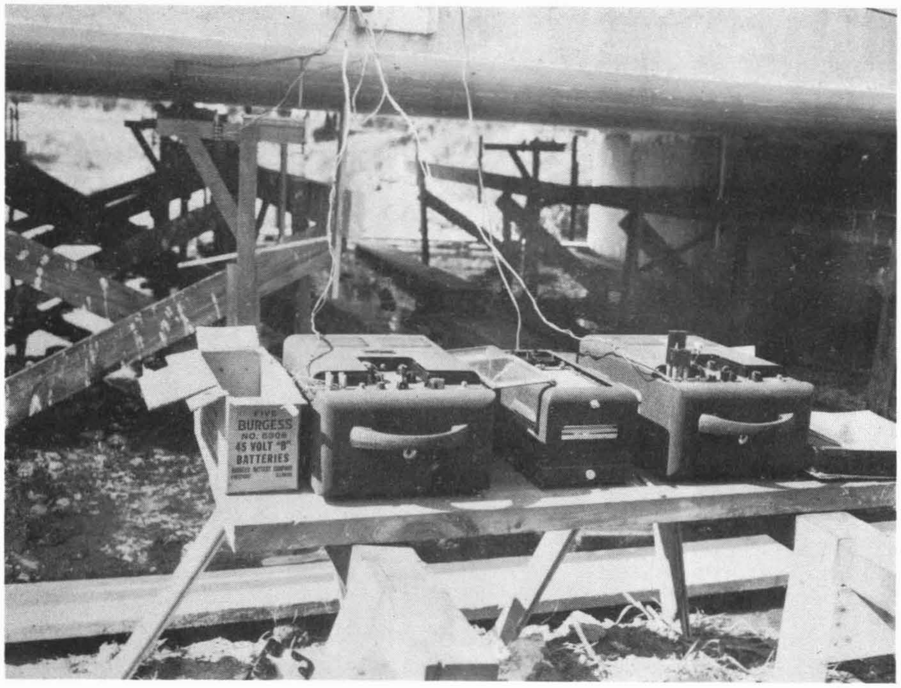


Fig. 8 Brush Equipment for the Dynamic Test

Fig. 9 Scale Truck. Total Weight 37,600 lbs.

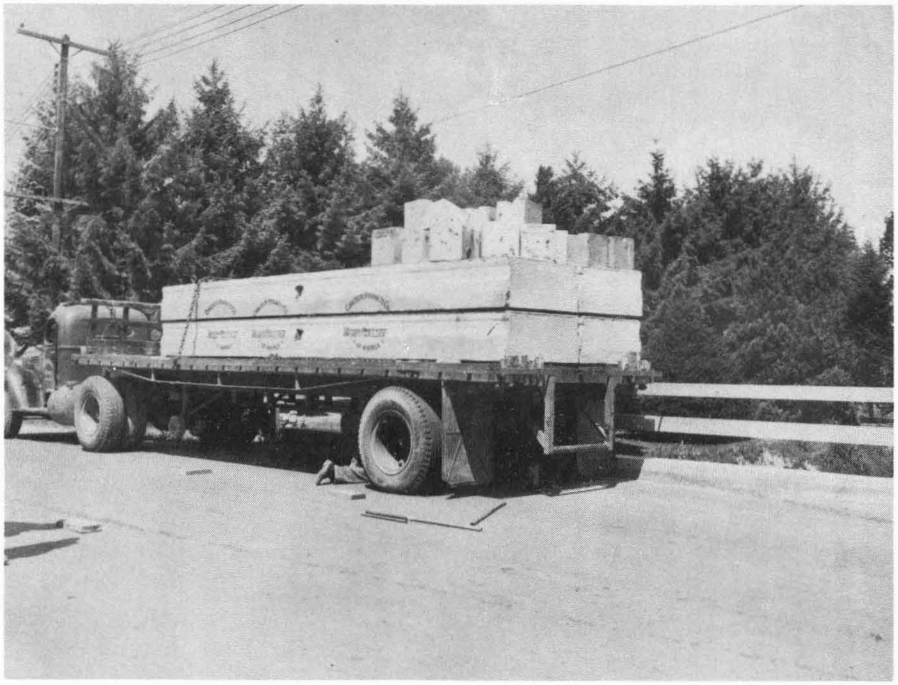


Fig. 10 Tractor Trailer Truck, loaded with Bridge Members and Movable Weight from Scale Truck. Total Weight of Rear Axle 47,700 lbs.

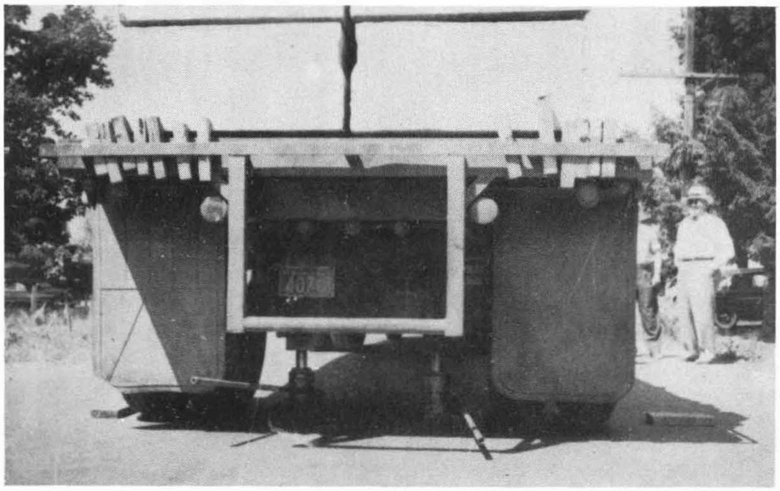
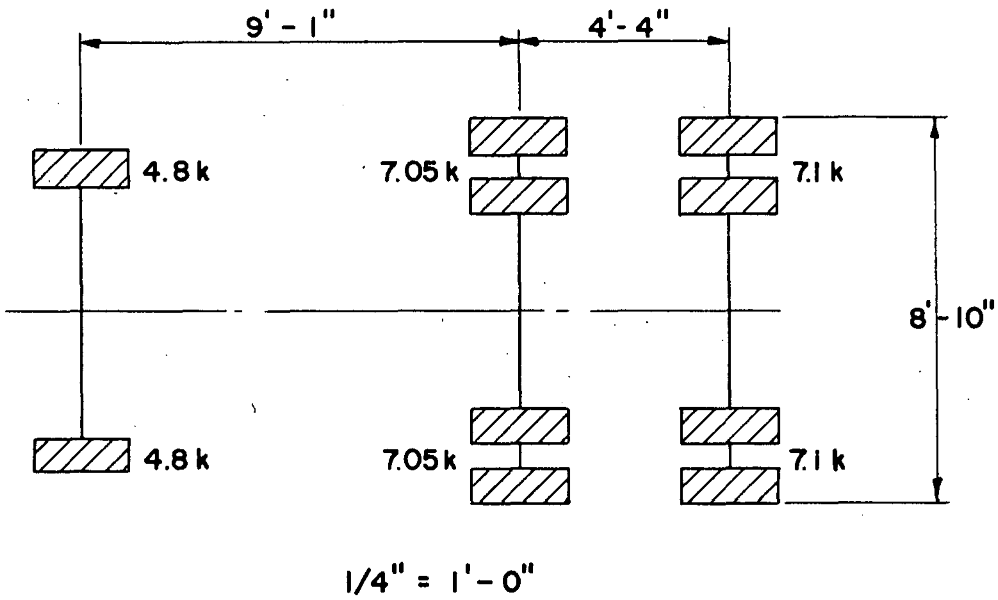
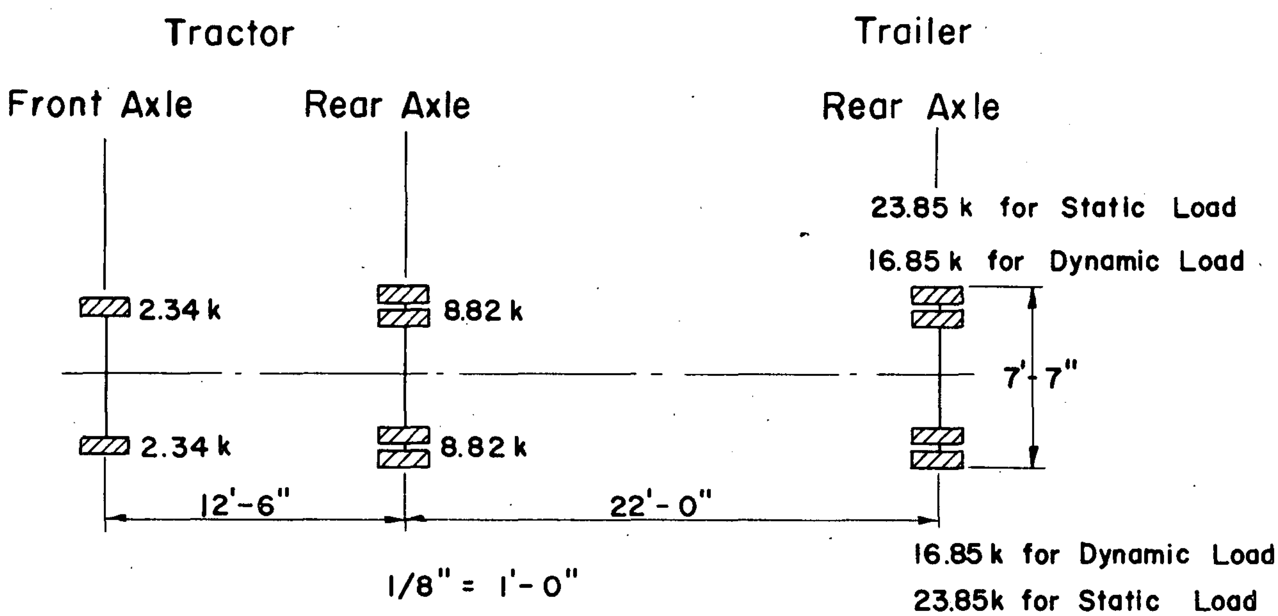


Fig. 11 Rear Axle of Trailer Jacked Up to Transfer Load of 47,700 lbs. on Single Beam of Bridge

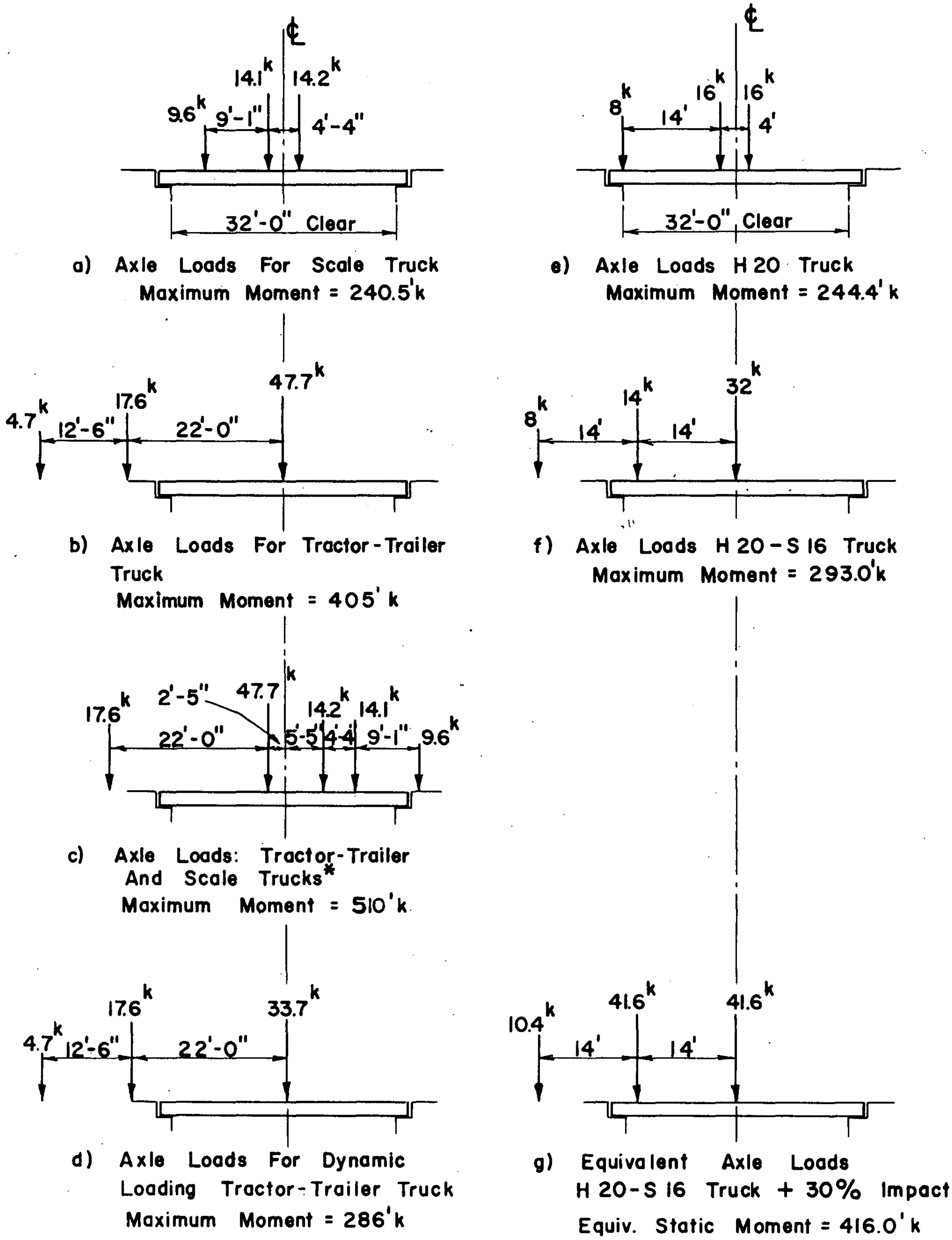


(a) Scale Truck



(b) Tractor-Trailer Truck

Fig. 12 Dimensions and Axle Loads of Trucks



a) Axle Loads For Scale Truck
Maximum Moment = 240.5'k

e) Axle Loads H 20 Truck
Maximum Moment = 244.4'k

b) Axle Loads For Tractor-Trailer Truck
Maximum Moment = 405'k

f) Axle Loads H 20-S 16 Truck
Maximum Moment = 293.0'k

c) Axle Loads: Tractor-Trailer And Scale Trucks*
Maximum Moment = 510'k

d) Axle Loads For Dynamic Loading Tractor-Trailer Truck
Maximum Moment = 286'k

g) Equivalent Axle Loads H 20-S 16 Truck + 30% Impact
Equiv. Static Moment = 416.0'k

* See Also Fig. 24

FIG. 13 Standard H 20 And H 20-S 16 Lane Loadings Compared With Lane Loadings For Test Trucks

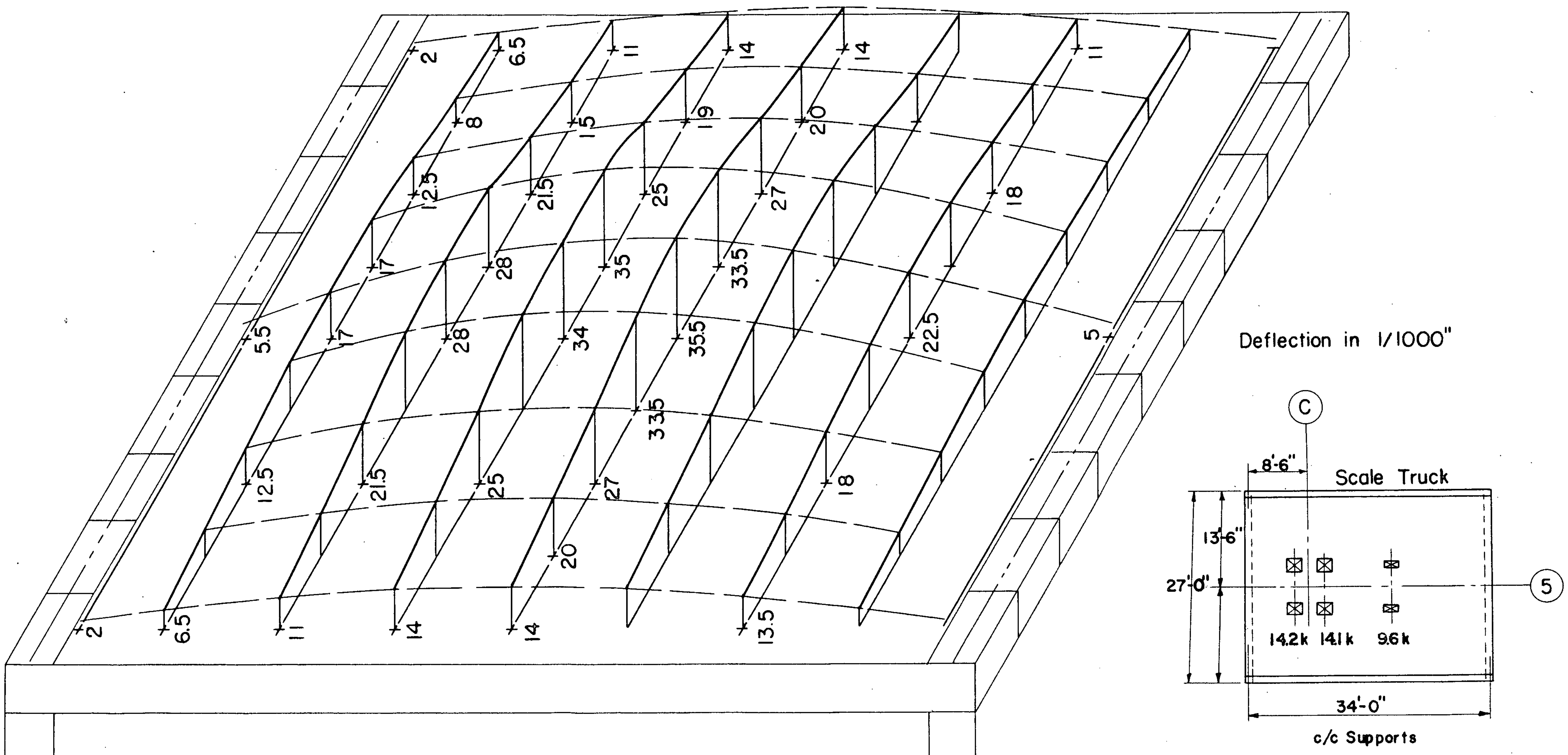


FIG. 14 DEFLECTION VALUES FOR LOAD POSITION AS SHOWN

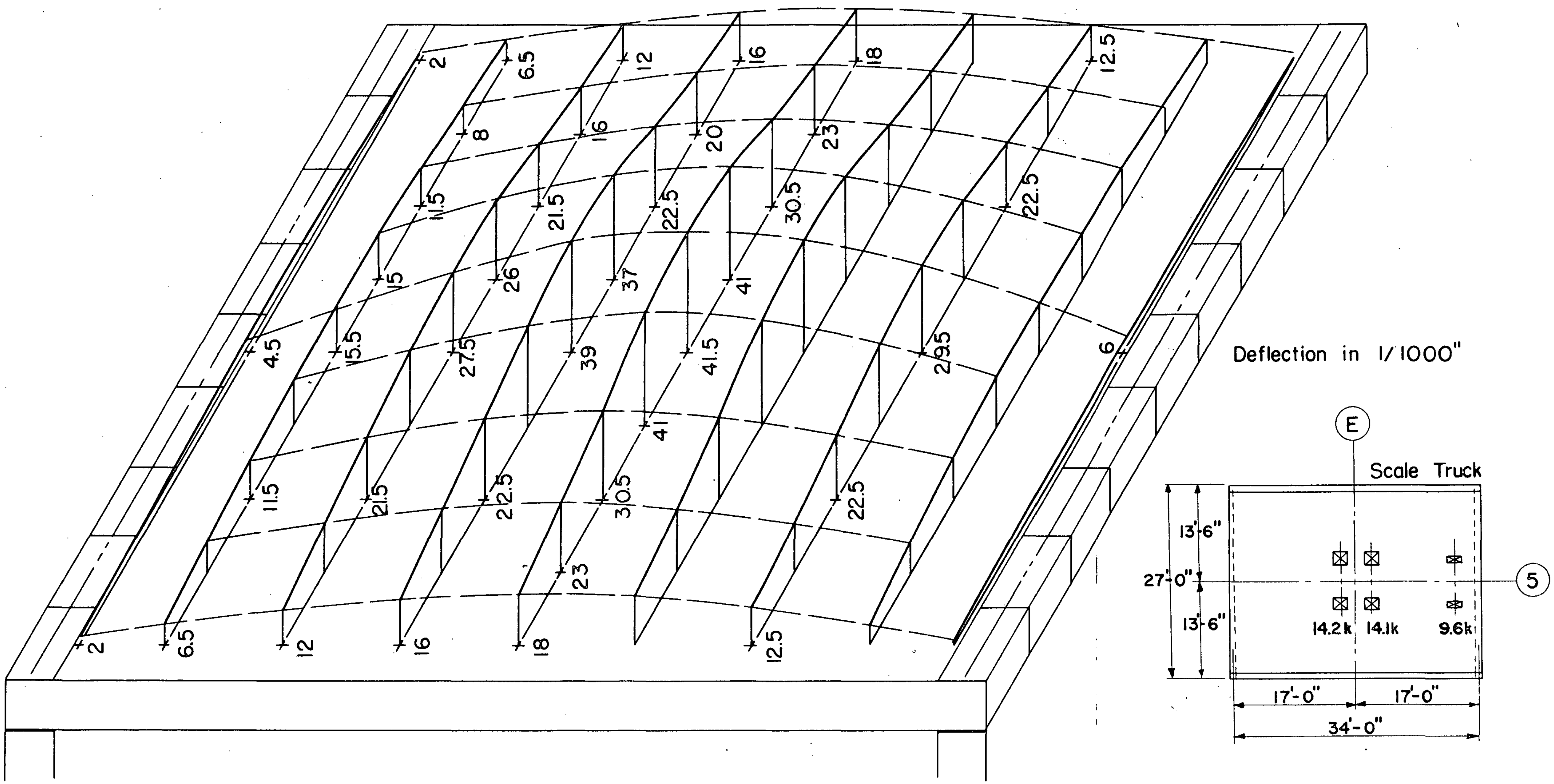


FIG. 15 DEFLECTION VALUES FOR LOAD POSITION AS SHOWN

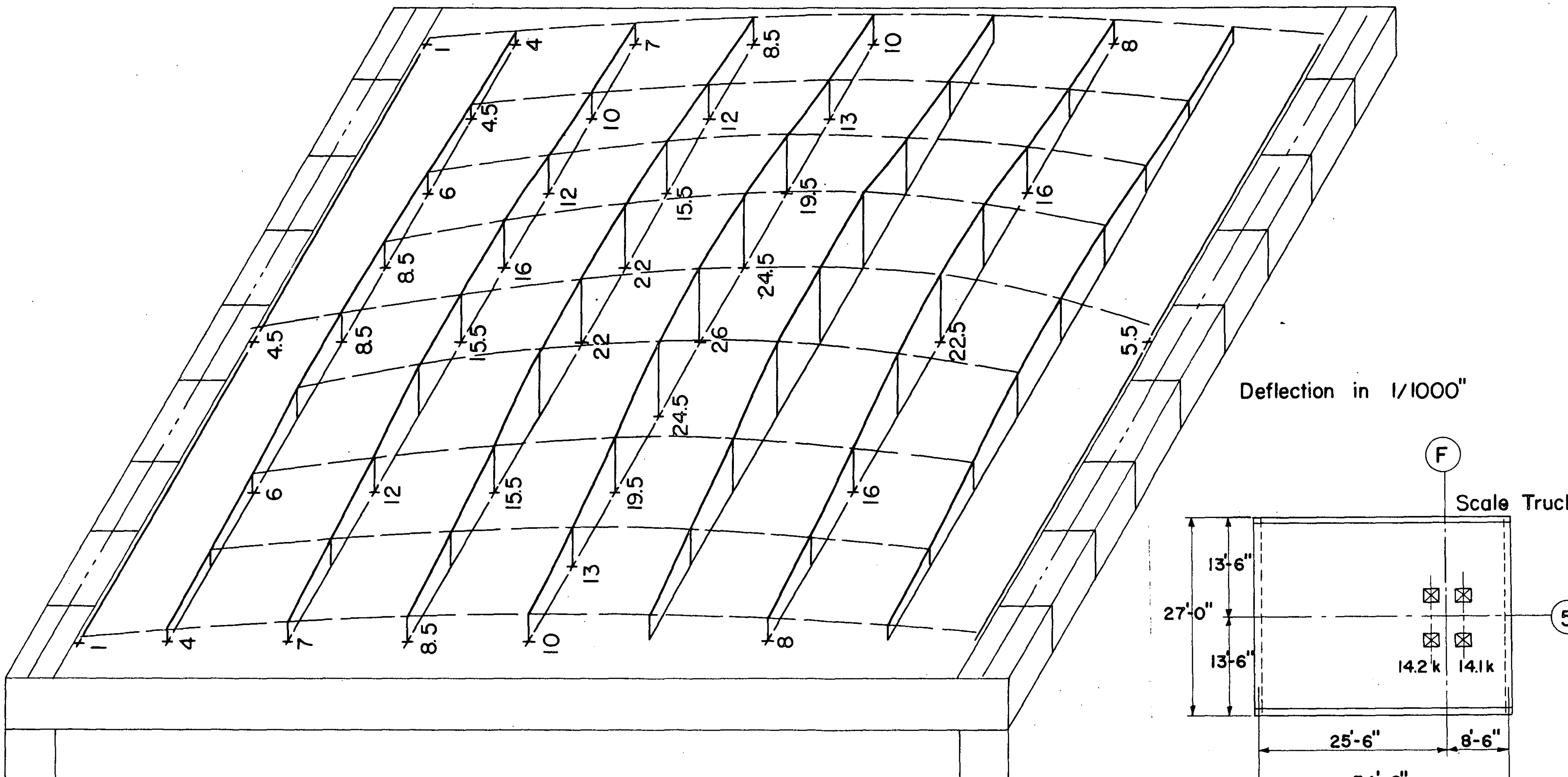
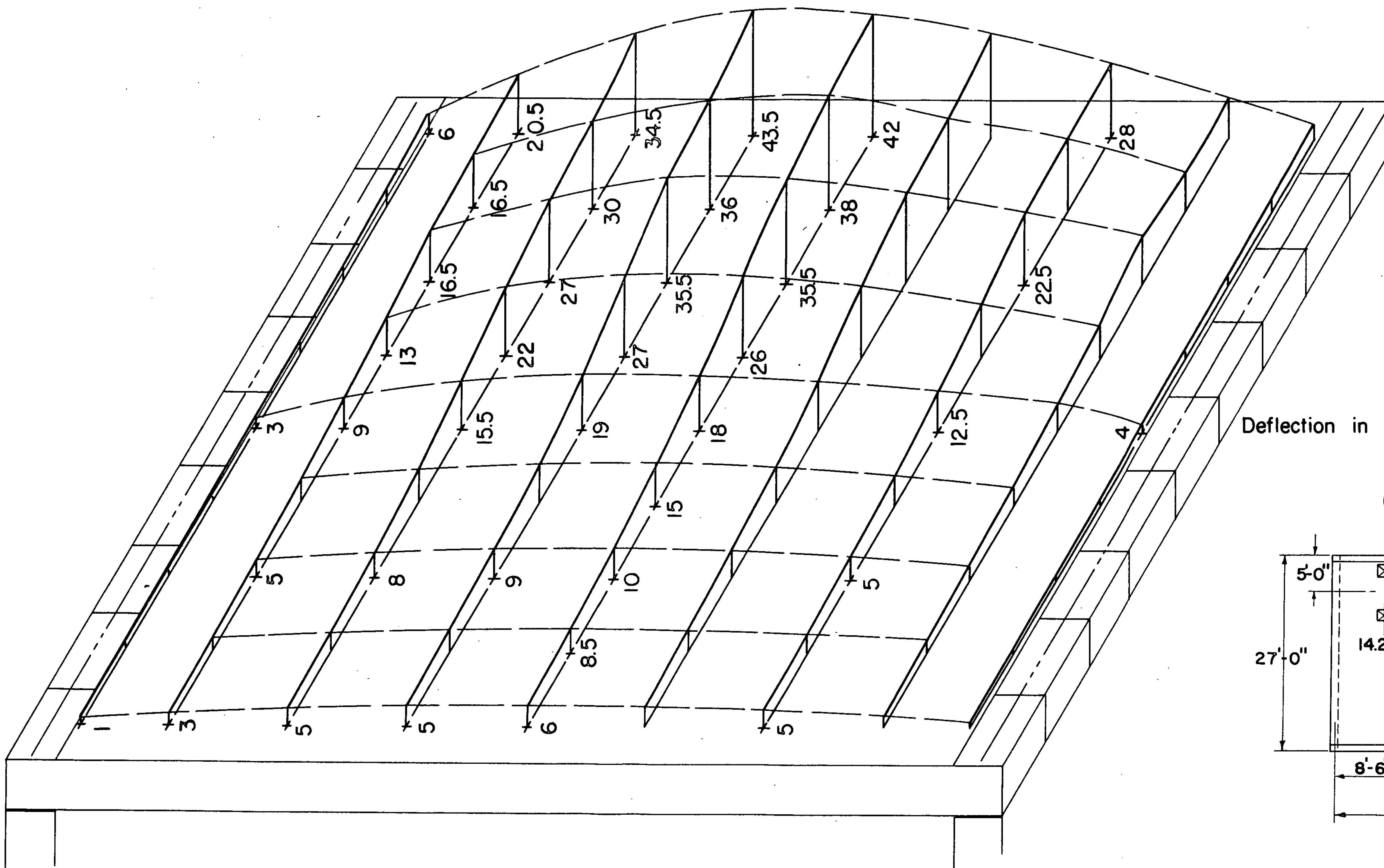


FIG. 16 DEFLECTION VALUES FOR LOAD POSITION AS SHOWN



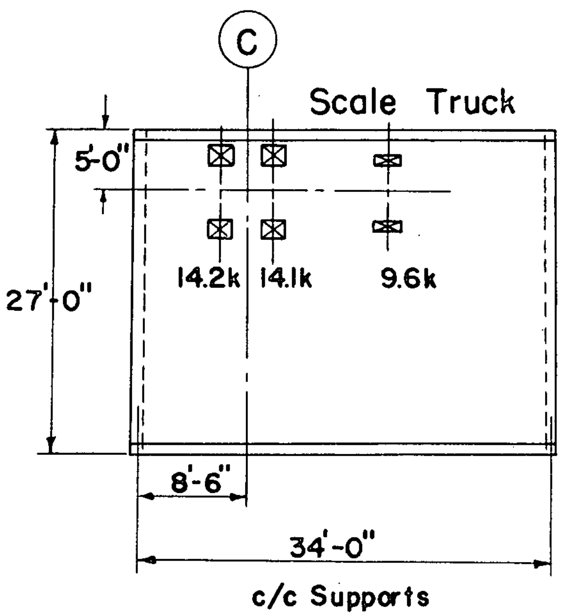
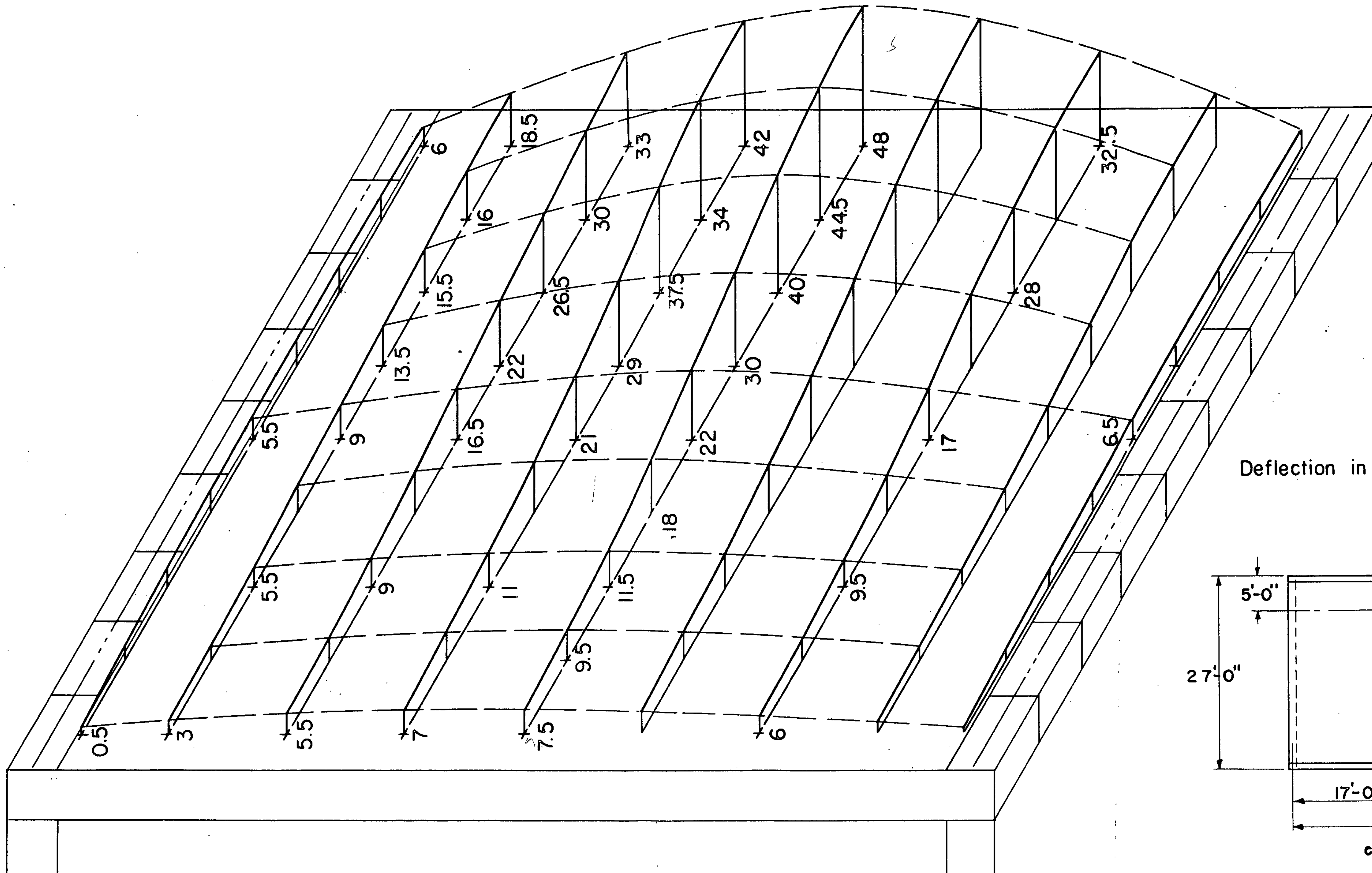


FIG. 17 DEFLECTION VALUES FOR LOAD POSITION AS SHOWN



Deflection in 1/1000"

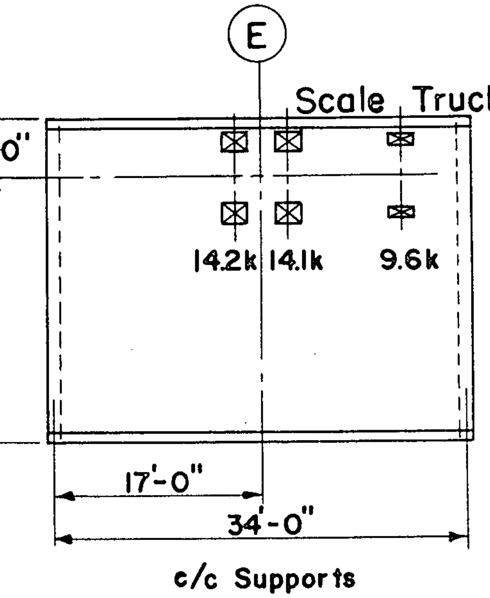


FIG. 18 DEFLECTION VALUES FOR LOAD POSITION AS SHOWN

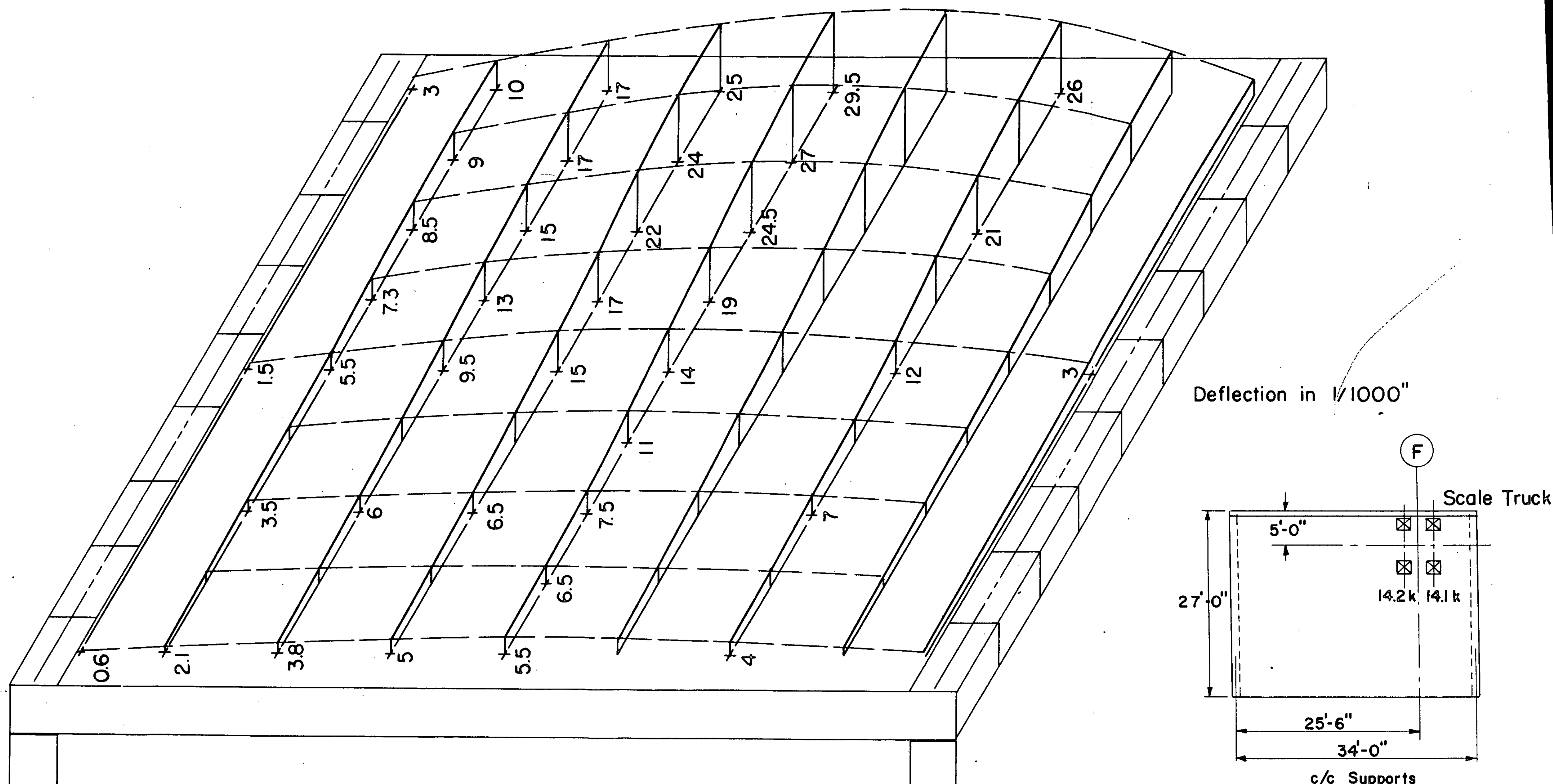


FIG. 19 DEFLECTION VALUES FOR LOAD POSITION AS SHOWN

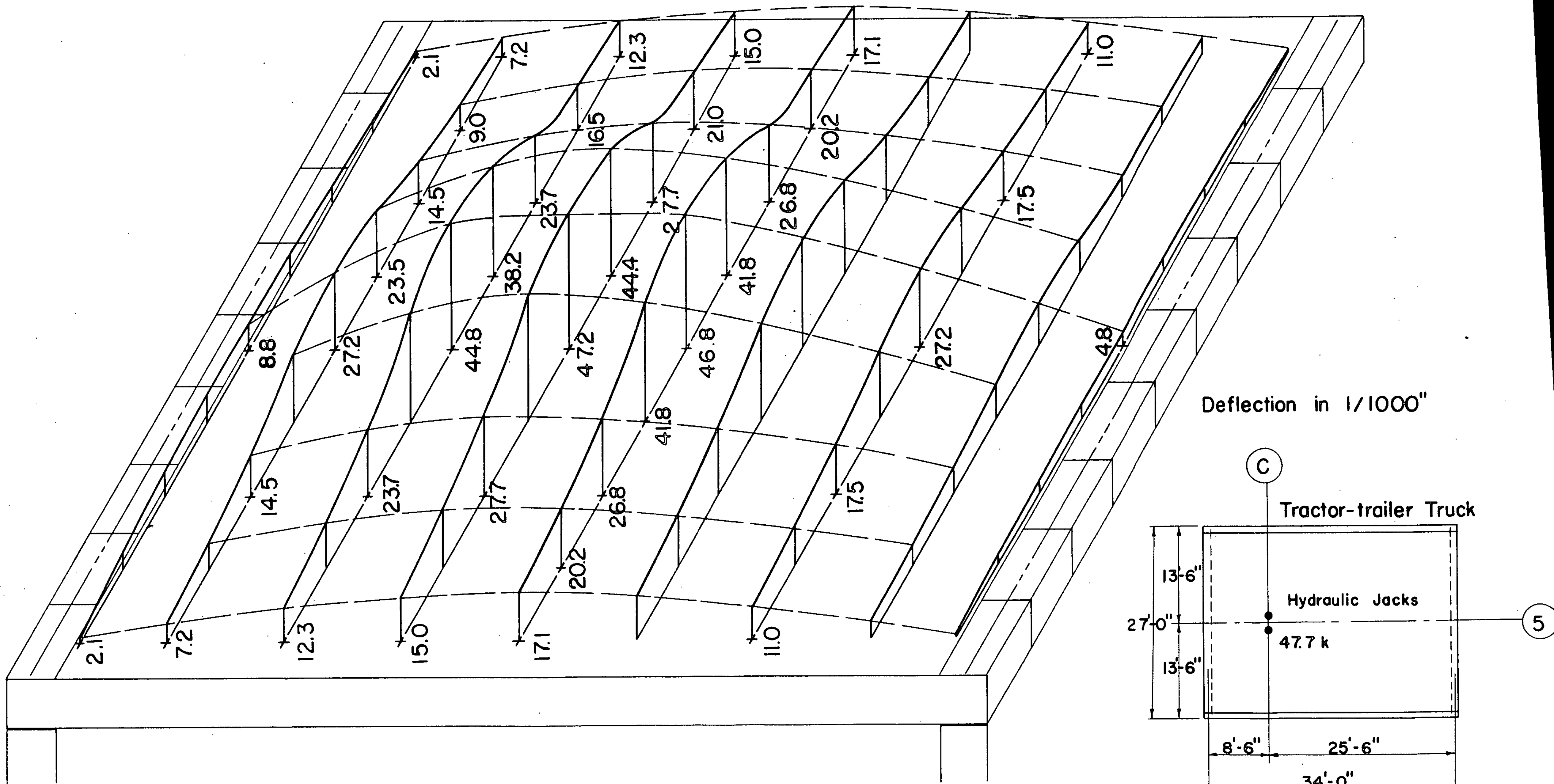


FIG. 20 DEFLECTION VALUES FOR LOAD POSITION AS SHOWN

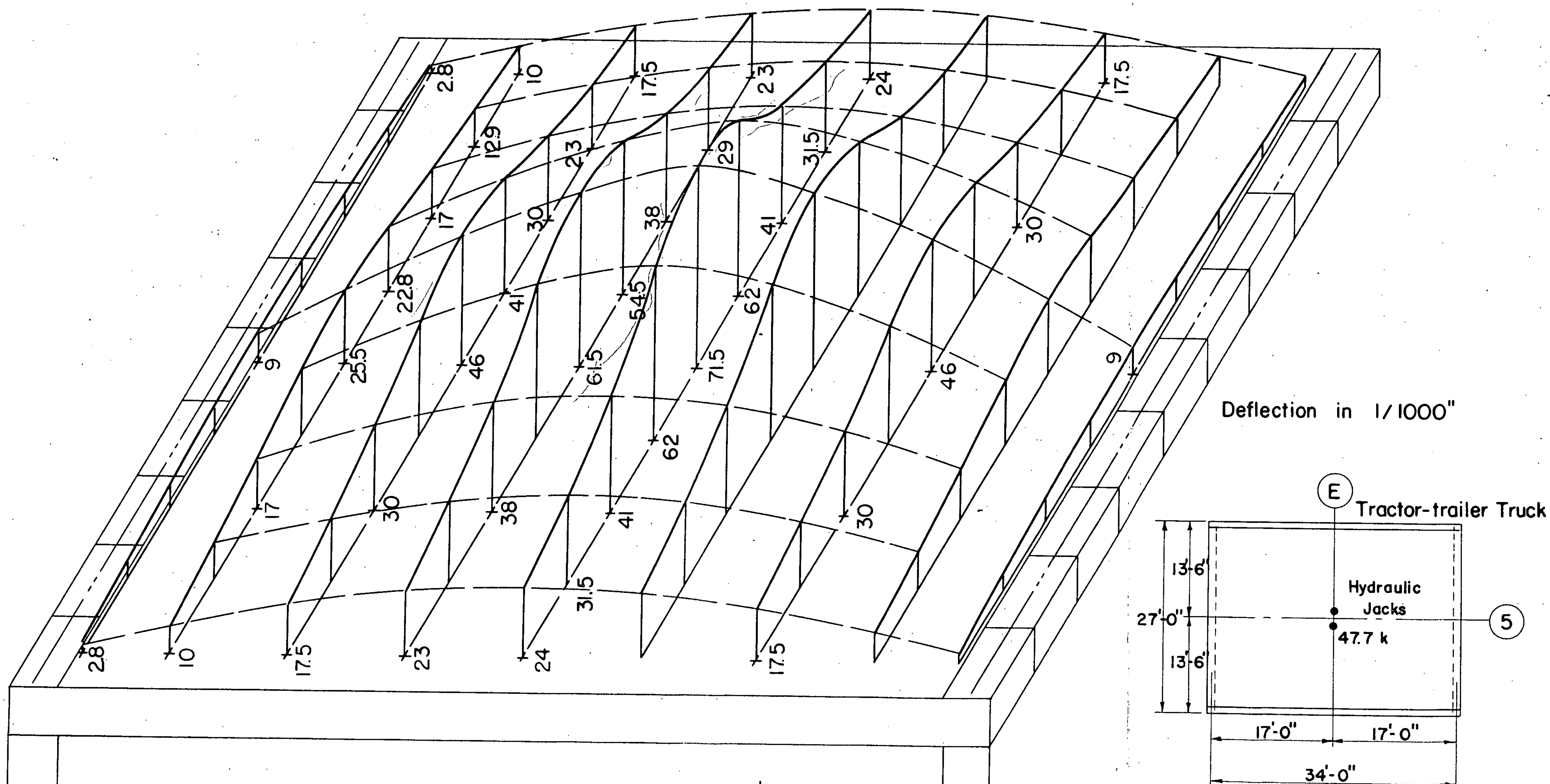


FIG. 21 DEFLECTION VALUES FOR LOAD POSITION AS SHOWN

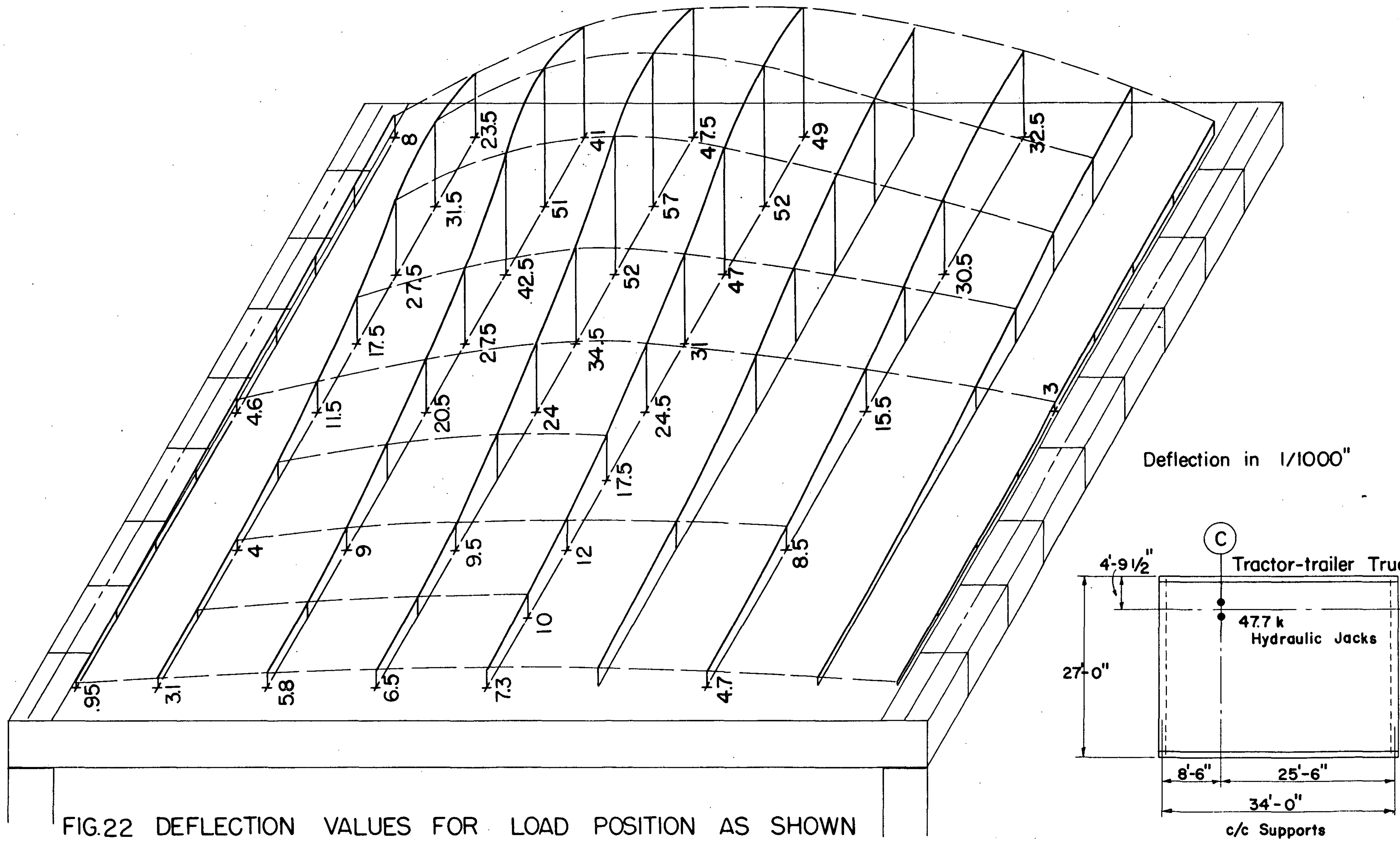
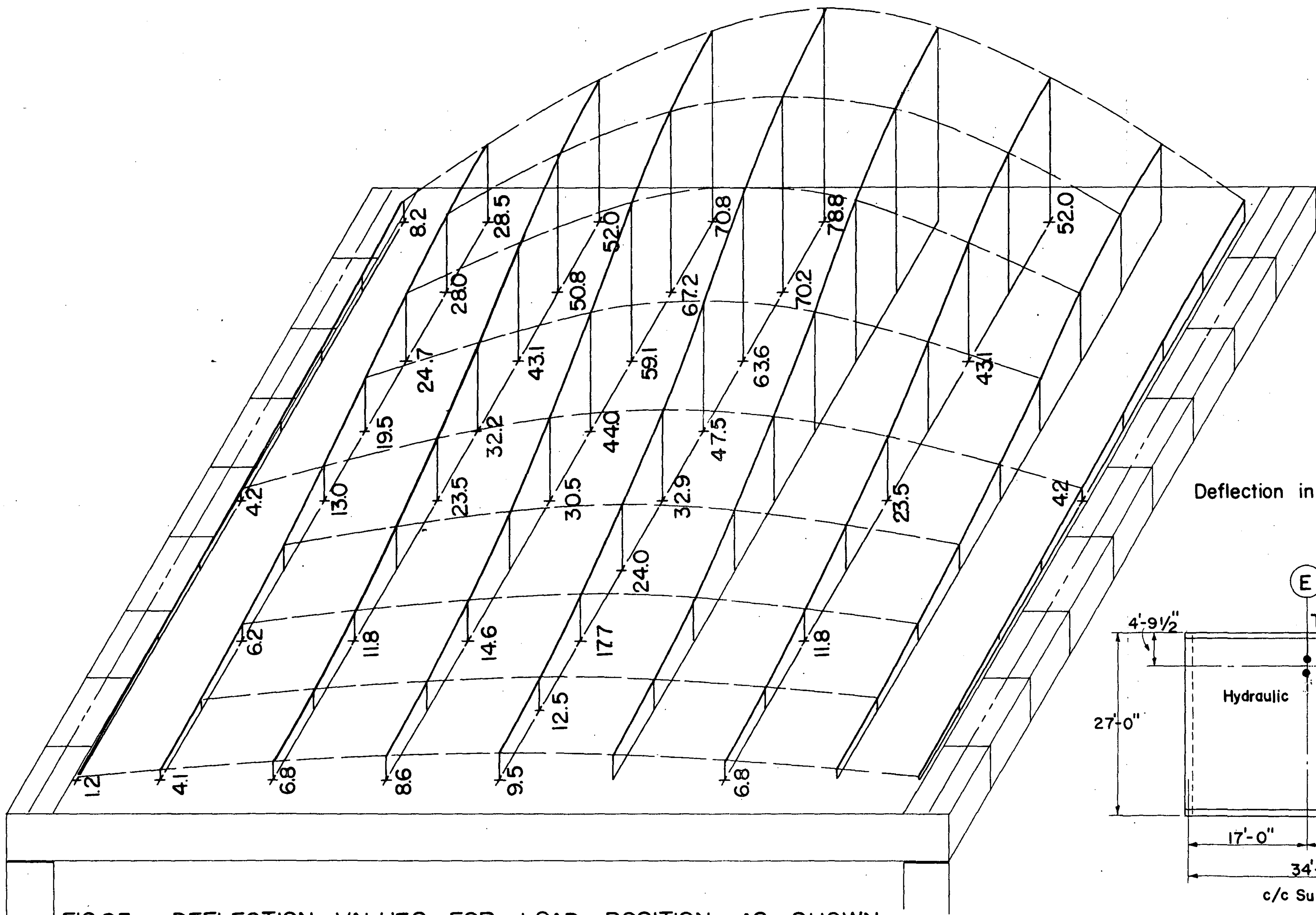


FIG. 22 DEFLECTION VALUES FOR LOAD POSITION AS SHOWN



Deflection in 1/1000"

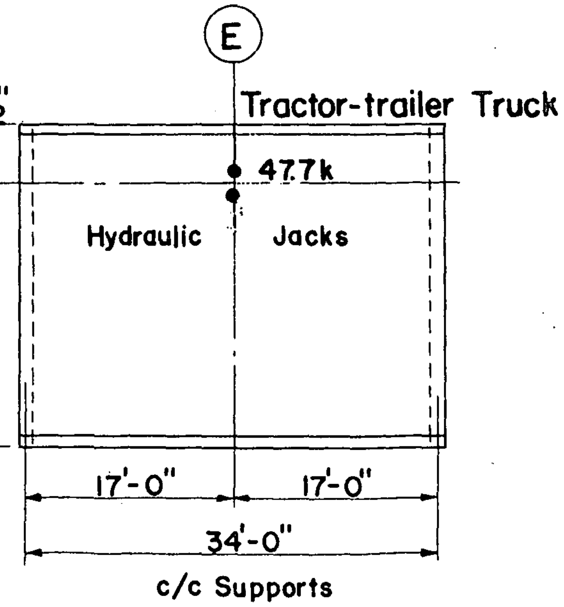


FIG. 23 DEFLECTION VALUES FOR LOAD POSITION AS SHOWN

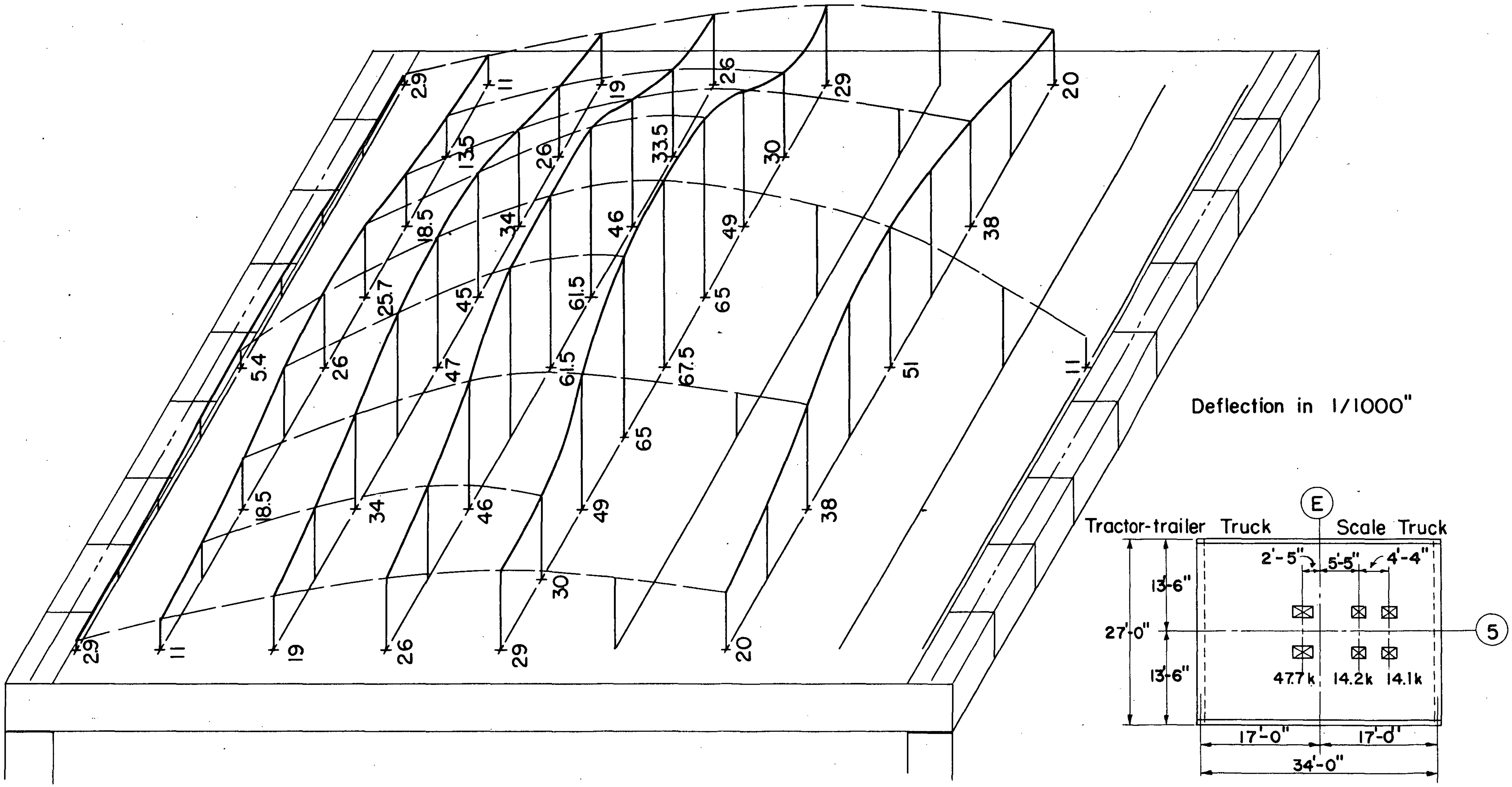


FIG. 24 DEFLECTION VALUES FOR LOAD POSITION AS SHOWN

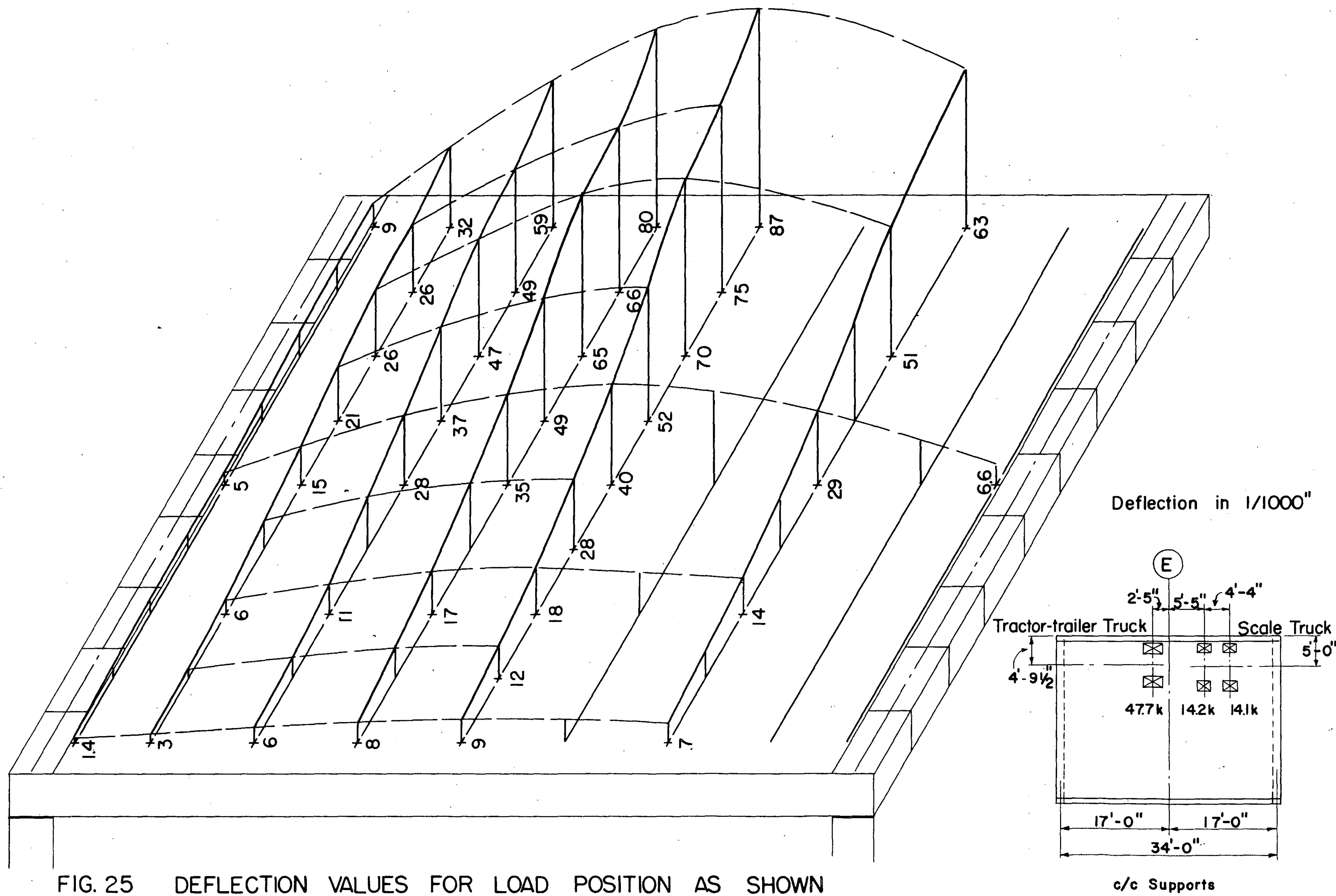
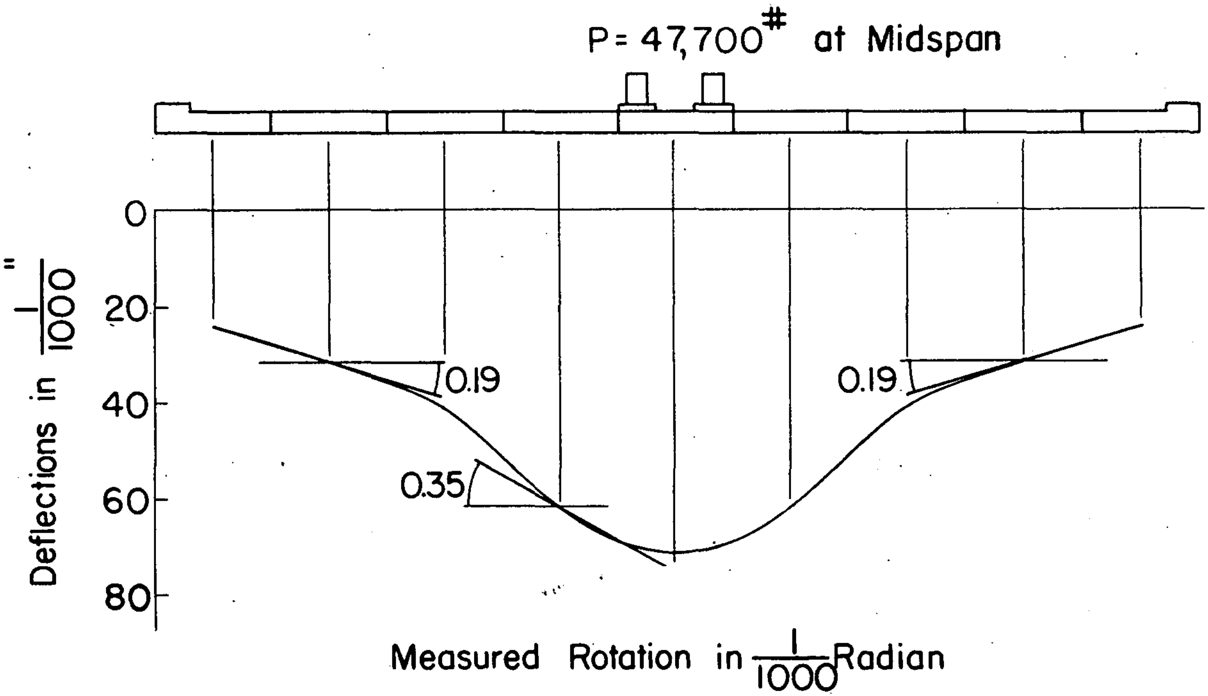
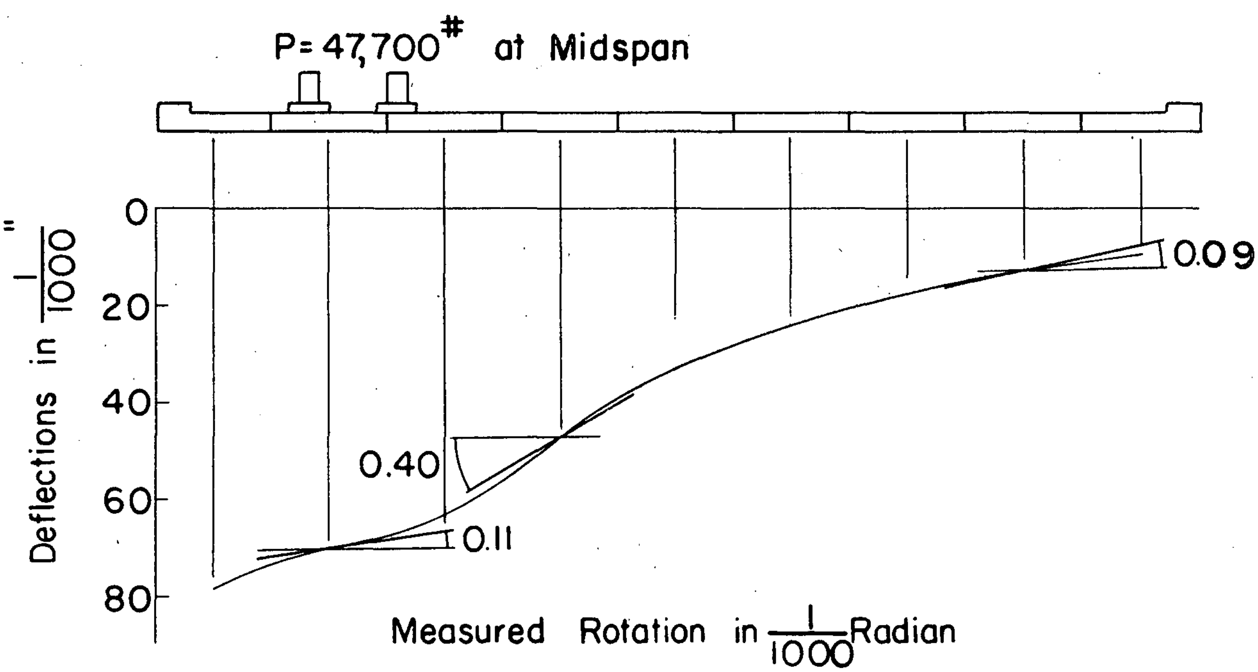


FIG. 25 DEFLECTION VALUES FOR LOAD POSITION AS SHOWN



a) For Concentrated Load On Center Beam



b) For Concentrated Load In Edge Lane

Fig. 26 Deflections and Torsional Rotations of Beams at Midspan

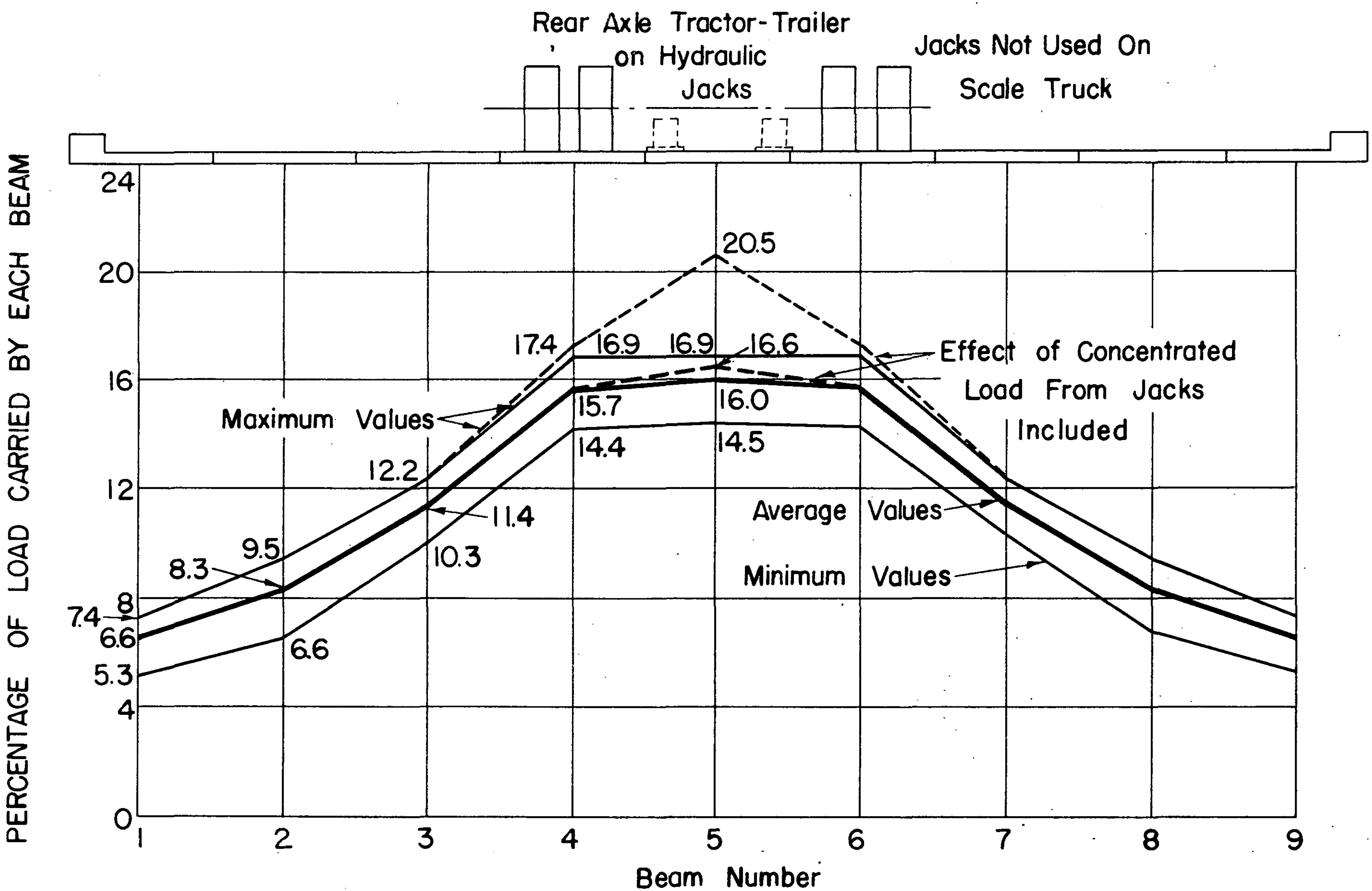


FIG. 27 LATERAL LOAD DISTRIBUTION FROM DEFLECTIONS

TRUCK IN CENTER LANE

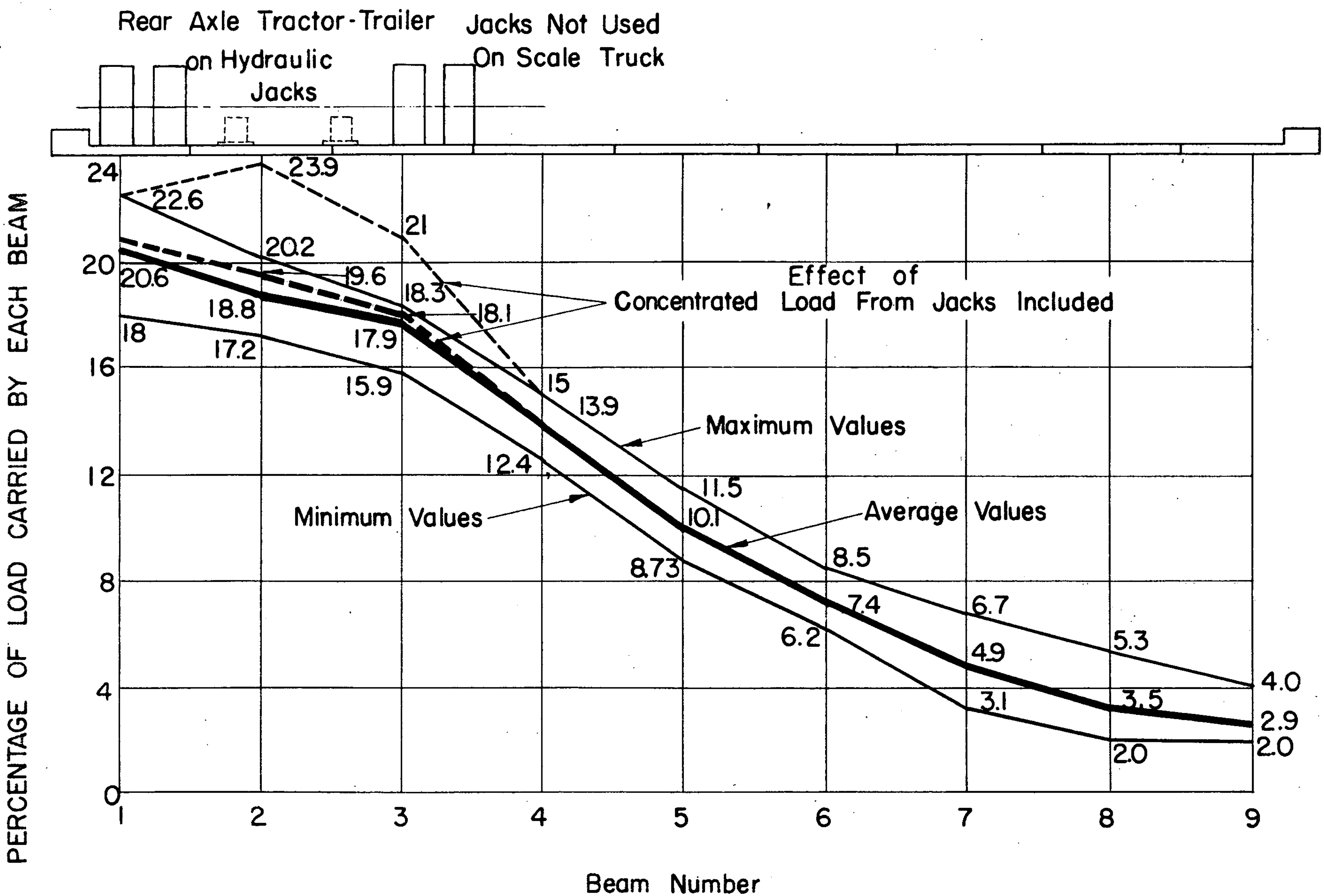
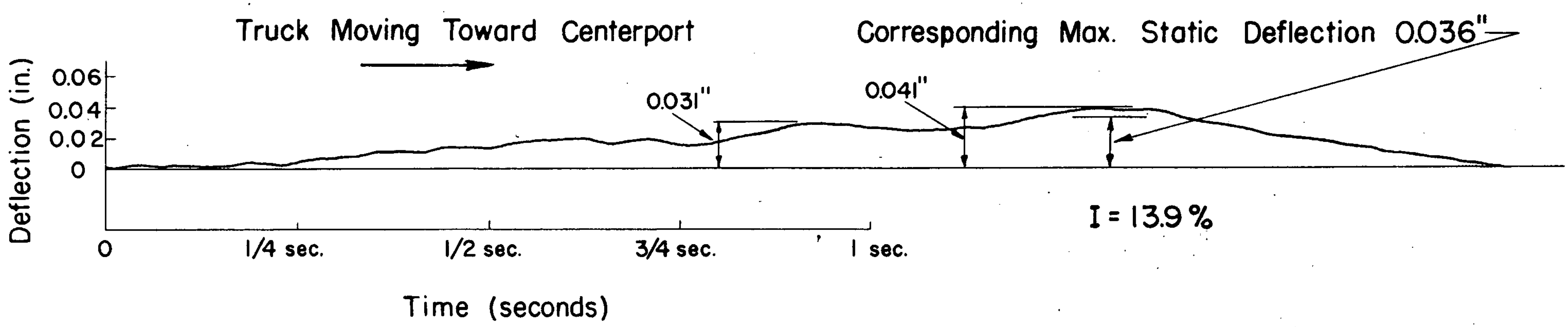
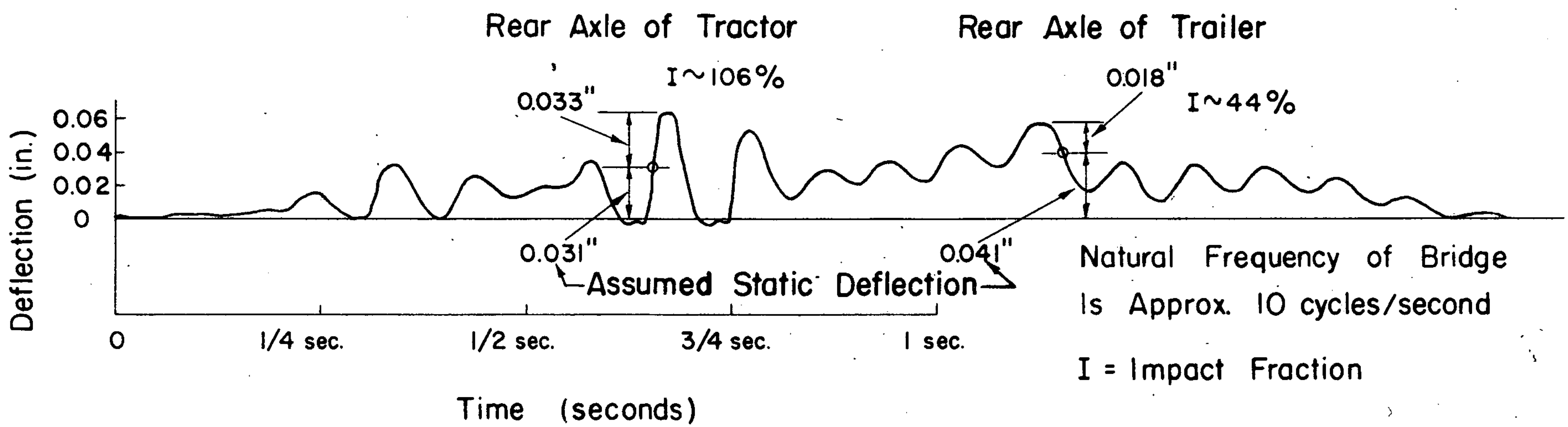


FIG. 28 LATERAL LOAD DISTRIBUTION FROM DEFLECTIONS

TRUCK IN EDGE LANE

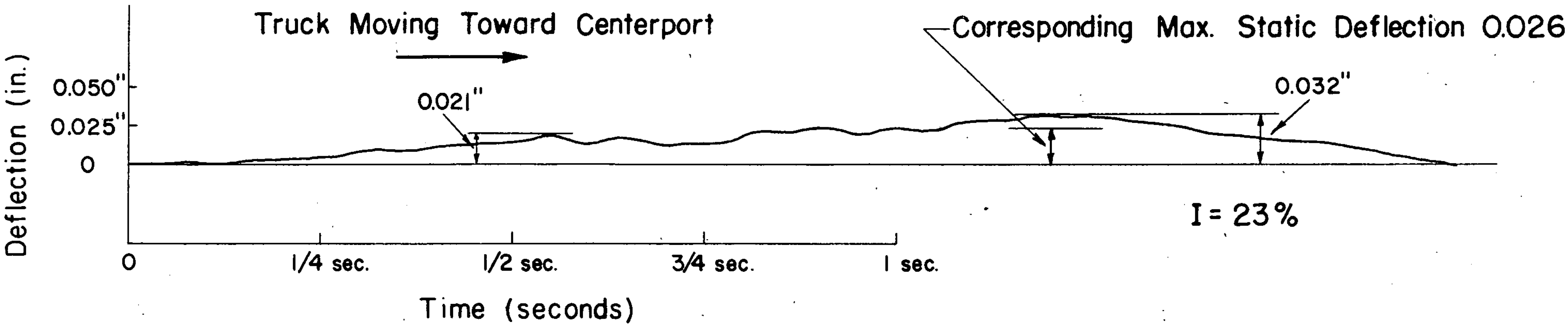


a) Tractor Trailer Truck Moving At 25 mph In Center Lane

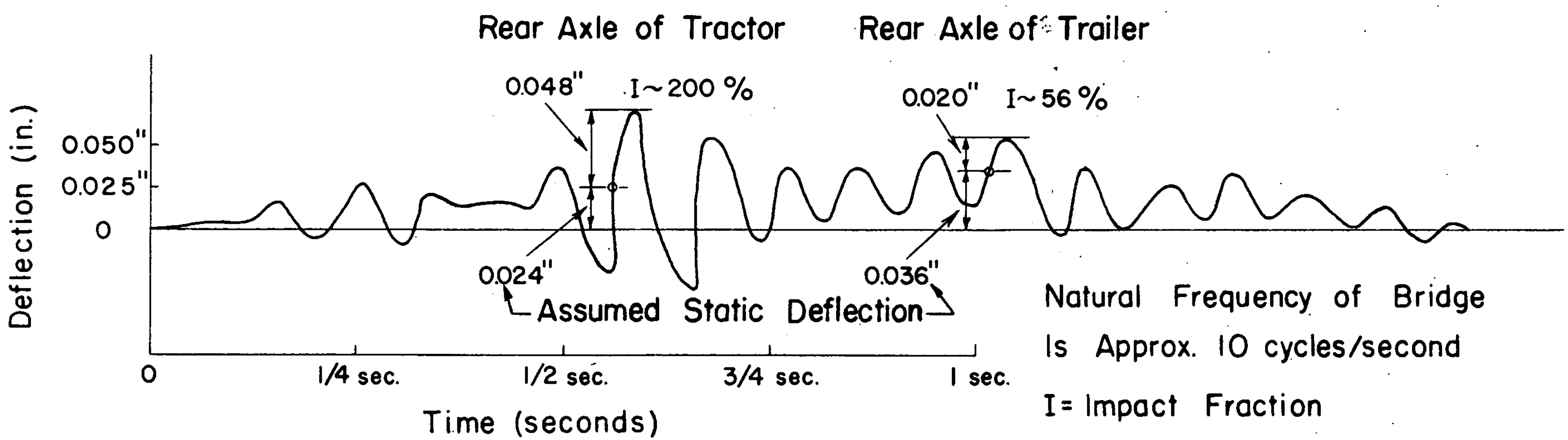


b) Tractor Trailer Truck Running Over Two Inch Plank At Midspan

Fig.29 Dynamic Deflection At Midspan Of Center Beam



a) Tractor Trailer Truck Moving At 25mph In Downstream Lane



b) Tractor Trailer Truck Running Over Two Inch Plank At Midspan

Fig.30 Dynamic Deflection At Midspan Of Downstream Edge Beam

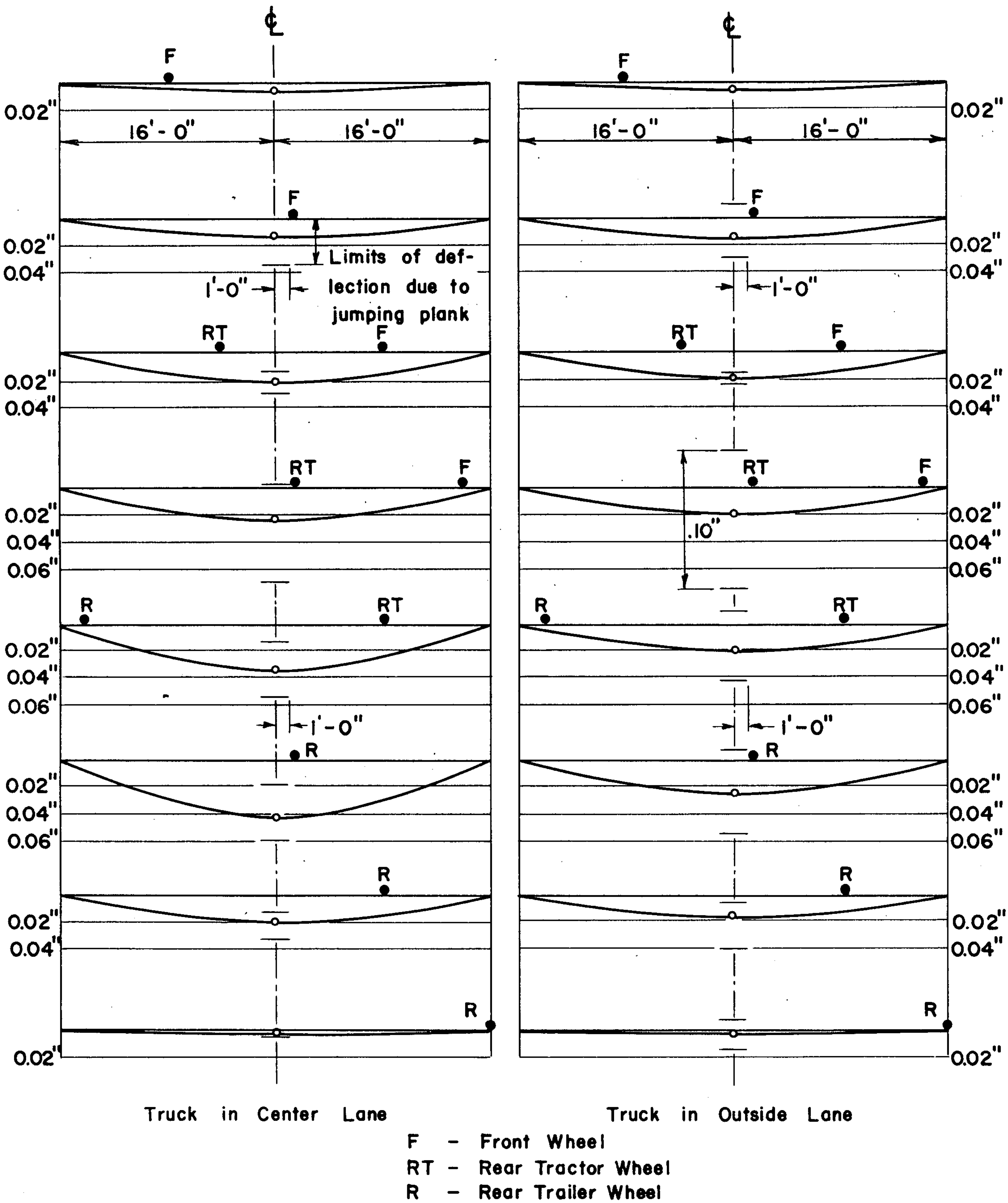


FIG. 31 Vertical Deflections At Midspan Caused By Truck Running Over Bridge At Approximately 25 mph

DATE DUE