UNIVERSITY OF ILLINOIS BULLETIN
PUBLISHED WEEKLY BY THE UNIVERSITY
Vol. X JANUARY 13, 1913 No. 16
[Re-entry at the Post Office at Urbana, Ill., as second-class matter under Act of Congress of July 16, 1894, pending]

BULLETIN NO. 64

TESTS OF REINFORCED CONCRETE BUILDINGS UNDER LOAD

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UNIVERSITY OF ILLINOIS
ENGINEERING EXPERIMENT STATION

URBANA, ILLINOIS

PRICE: FIFTY CENTS
EUROPEAN AGENT
CHAPMAN AND HALL LTD., LONDON

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ENGINEERING EXPERIMENT STATION

BULLETIN NO. 64 JANUARY, 1913

TESTS OF REINFORCED CONCRETE BUILDINGS UNDER LOAD

BY ARTHUR N. TALBOT, PROFESSOR OF MUNICIPAL AND SANITARY ENGINEERING AND IN CHARGE OF THEORETICAL AND APPLIED MECHANICS, AND WILLIS A. SLATER, FIRST ASSISTANT IN ENGINEERING EXPERIMENT STATION

CONTENTS

LIST OF TABLES

LIST OF FIGURES

TESTS OF REINFORCED CONCRETE BUILDINGS UNDER LOAD

I. Introduction.

1. *Preliminary.*—In the development of the newer types of building construction, the need of further information on the action of the structure in its various parts has been felt. Analysis gives methods of calculation of stresses and laboratory tests give data on the action of individual members; but the truth of the assumptions used in analysis may be questioned, and because of the method of fabrication or the influence of one part on another the action of the structure may not be in exact accord with the conclusions derived from analytical considerations. It is especially important that knowledge on the amount and distribution of deformations and stresses actually developed in structures be extended, and every effort may well be made to determine these stresses by tests of structures themselves. Many load-deflection tests of structures have been made, and such tests are required by city building departments as a condition of acceptance for allowable loading, and these tests have been used by construction companies and engineers to demonstrate the adequacy of various designs. Load-deflection tests are of value in judging of the quality of the workmanship and in giving confidence in the structure, but they throw little light on the stresses developed in the different parts or upon their distribution. The deflections observed in such tests constitute a very inadequate measure of the stresses and may even be misleading in this respect. Slight deflections, which have been taken to indicate low stresses in steel and concrete, may actually be accompanied by high stresses. In the matter of design there has been a divergency of views on the relation between the bending moment at a section at the support and that at the middle of the beam, on the distribution of stresses across a flat slab acting as the flange of a T-beam, on the restraint of girders and beams, and on the stresses developed in the flat slab type of floor construction. It is evident that measurements of the deformation in structures may be expected to greatly assist the settlement of such questions as these.

The measurement of deformation in the various parts of a structure by a field test is a recent development in testing work. It may be expected that in the early stages of the development of such field tests difficulties will be encountered and that experience will bring out the methods which are most satisfactory and will indicate the precautions which must be observed to insure accurate and trustworthy results. The statement of the requirements for such a test will be of value in making other tests, and the methods of course should be carefully stated with the record of such tests.

2. *Scope of Bulletin.*—This bulletin records the results of three field tests made on reinforced concrete floor systems in which the measurement of deformations or strains in the parts of the structure was an important feature. As these tests comprise the earliest known measurements of this kind made upon reinforced concrete buildings and as the writers have been connected with the development of this method of testing, it has seemed proper to include a discussion of the method of testing—the use of the instruments, the methods of observation, the precautions to be taken, the accuracy of the results and the methods of loading. The bulletin then gives a record of the results of the tests on the floor systems of two buildings of the beam and girder type and of one building of the flat slab type, and contains discussions of the stresses developed and the general phenomena observed.

3. *Acknowledgment.*—The technical part of making the tests was done as the work of the Engineering Experiment Station of the University of Illinois. The first building test in which deformations of steel and concrete were measured was made on the Deere and Webber Building in November, 1910. This test was under the direct supervision of Mr. Arthur R. Lord, then Research Fellow in the Engineering Experiment Station. Mr. Lord is entitled to much credit for his work in directing this test and for the initiative, foresight and care used in developing methods and in making the test. The report of the test on the Deere and Webber Building and the discussion of the results were prepared by Mr. Lord and with his permission are included in this bulletin. Mr. W. A. Slater was in direct supervision of the test of the Wenalden Building and the Turner-Carter Building, and has been intimately connected with the other tests of the kind named in this bulletin, and to him credit is due for many of the methods and details of the testing work and for formulating the provisions and precautions necessary to give accuracy and trustworthiness to the results.

The tests were undertaken as co-operatove work. The tests on the Wenalden Building and the Turner-Carter Building were made in connection with the Committee on Reinforced Concrete and Building Laws of the National Association of Cement Users, and the president and the treasurer of the Association raised the funds to defray expenses of the test. The contractors who erected the buildings also assisted in these

tests. The expense of the Deere and Webber test was borne by the building contractor, to whom especial credit should be given for very active interest and co-operation in initiating a new line of tests.

The tests were conducted by members of the staff of the Engineering Experiment Station of the University of Illinois. These included Messrs. H. F. Moore, W. A. Slater, A. R. Lord, D. A. Abrams, N. E. Ensign and H. F. Gonnerman. The observers on the Deere and Webber Building test were Messrs. Moore, Slater and Lord; on the Wenalden Building test, Messrs. Moore, Slater and Ensign; and on the Turner-Carter Building, Messrs. Moore and Slater. Professor Talbot was in charge of the work. Papers covering much of the ground of this bulletin have been presented before the National Association of Cement Users and published in Vols. VII and VIII of the Proceedings of the Association.

4. *Comment.*—A few words on the basis and limitations of such tests may not be out of place here. It must be borne in mind that the measurements and observations are subject to some uncertainty as compared with certain laboratory tests; they are not exact or precise, and some erratic readings may be expected. The measuring instrument is used under unfavorable conditions. The gauge holes are deep in the concrete and the measurements may be interfered with by dust or other obstructing matter. It is evident that great care and much skill is necessary in making observations. Each test made has shown advances in accuracy and certainty, and further experience ought to show additional progress. Besides, it must be understood that the structure itself is not entirely homogeneous and that all parts of it do not act alike. Further, the structure itself is tied together so closely that stress in one portion may be modified or assisted in an unknown amount by another portion, which may not be thought to affect it. The modulus of elasticity of the concrete in the structure is not easily determined. The load-deformation diagrams may be irregular and imperfect. This all means that care must be taken in the interpretation of results and that some irregularities and uncertainties must be expected. With careful work important information will be brought out, as these tests show, and an accumulation of data on the action of structures, and tests of special features of construction will advance knowledge of structural action and be worth many times the cost of the work.

II. The Testing of Buildings.

5. *Building Tests Made.*—The number of building tests in which deformations have been measured is comparatively small. A list is here given of all known tests on reinforced concrete building floors in which

deformations in the steel and the concrete have been measured. The methods used in all these tests are essentially the same; they have been developed at the University of Illinois Engineering Experiment Station. Fig. 1 shows the range in size of the test areas in the buildings tested.

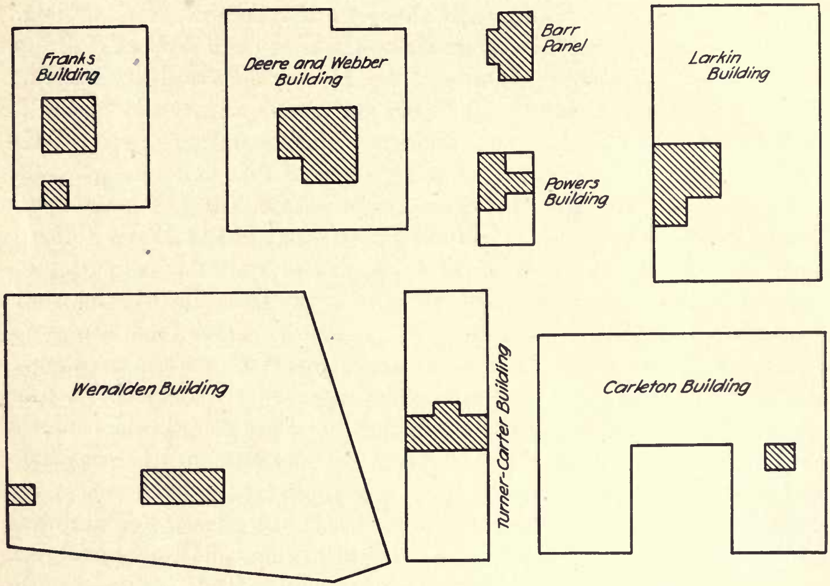

FIG. 1. DRAWING SHOWING RELATIVE SIZES OF TESTS.

Tests have been made on steel structures by the U. S. Bureau of Standards using methods somewhat similar to those here described, but as this bulletin is concerned primarily with the results of tests on reinforced concrete floor systems specific mention of the tests by the Bureau of Standards is omitted.

Test No. 1. Deere and Webber Building, Minneapolis, Minnesota, October and November, 1910. Flat slab floor with four-way reinforcement, built by Leonard Construction Company of Chicago, and tested by co-operation between the contractors and the Engineering Experiment Station of the University of Illinois.

Test No. 2. Wenalden Building, Chicago, Illinois, June and July, 1911. Beam and girder building constructed by Ferro-Concrete Construction Company of Cincinnati, and tests made by co-operation between the National Association of Cement Users, the construction company, and the Engineering Experiment Station of the University of Illinois.

Test No. 3. The Powers Building, Minneapolis, Minnesota, July and

August, 1911. Flat slab floor with two-way reinforcement, built and tested by Corrugated Bar Company of St. Louis.

Test No. 4. Franks Building, Chicago, Illinois, August, 1911. Flat slab floor with four-way reinforcement, built and tested by Leonard Construction Company of Chicago. Prof. W. K. Hatt of Purdue University was employed as consulting engineer for this test.

Test No. 5. Turner-Carter Building, Brooklyn, New York, September, 1911. Beam and girder floor, built by Turner Construction Company of New York; test made by co-operation between National Association of Cement Users, the construction company, and the Engineering Experiment Station of the University of Illinois.

Test No. 6. Carleton Building, St. Louis, Missouri, October, 1911. Flat slab floor with two-way reinforcement, built and tested by Corrugated Bar Company.

Test No. 7. Barr Building, St. Louis, Missouri, December, 1911. Full size test panel (25 x 26 ft. 9 in.); terra-cotta tile used to lighten construction; consists of two-way T-beams with web between tile on tension side and concrete flange above the tile on the compression side. Panel built by the Corrugated Bar Co. to demonstrate efficiency of design proposed for Barr Building in St. Louis; test made by Corrugated Bar Company.

Test No. 8. Ford Motor Building, Detroit, Michigan, February and March, 1912. Flat slab floor, built and tested by the Corrugated Bar Company.

Test No. 9. Larkin Building, Chicago, Illinois, August, 1912. Flat slab floor, built and tested by Leonard Construction Company.

The Deere and Webber Building test was undertaken to learn of the general action of the flat floor slab. The tests on the Wenalden Building and the Turner-Carter Building were made to find the general action of the beam and girder type of construction. The tests made by the Corrugated Bar Company were for their own information but the results of the tests on the Powers Building and on the Barr Building test panel were presented before the Eighth Annual Convention of the National Association of Cement Users. Those of the Carleton Building and the Ford Motor Building were in the nature of investigation of special features of design. The Franks Building test made by the Leonard Construction Company was an investigation to obtain a basis for making provision in the Chicago building code for this form of construction. The Larkin test was the most extensive of those enumerated and was made with the object of furnishing the Leonard Construction Company with additional information for the design of flat slab floors.

6. *Definitions.*—In the following descriptions of tests, many terms will be used for which somewhat arbitrary definitions will need to be made. These definitions are given here:

Gauge Hole: A small hole (0.055 in. is here recommended) drilled into the steel bar or into the plug inserted in the concrete has been termed a gauge hole. It is for the admission of the point of a leg of the extensometer.

Gauge Line: The gauged length connecting a pair of gauge holes is termed a gauge line.

Reading: A reading is a single observation on any gauge line.

Observation: An observation as here used is the average of a number of readings.

Correction: A correction is the amount which if added algebraically to the observation will give the observation which would have been obtained if the instrument had not changed from its reference length.

Series of Observations: A set of observations on all gauge lines or on a selected number of gauge lines at a given load and taken in an established order is termed a series of observations.

Interval: An interval as used here is the time elapsing between consecutive observations, and all intervals in any series are (for lack of more exact information) assumed to be equal. For this purpose the average of two consecutive observations on standard gauge lines is considered a single observation.

Standard Gauge Line: Changes in the temperature of the instrument always occur in the course of a test. Frequently these changes are sufficient to cause an appreciable change in the length of the instrument. These and other small changes (usually unaccounted for) in the length of the instrument will introduce errors into the results unless the necessary corrections are applied to the observations. For the purpose of determining what these corrections should be, it is necessary to have reference to the standard bar may be understood to signify the standard constant as possible. This gauge line is termed a standard gauge line. Usually it is placed on a steel bar separate from the structure, and this has given rise to the term standard bar. In several of the tests, however, the standards have consisted of gauge lines placed in the steel and concrete of the structure remote from the area affected by the load. Standard gauge line is adopted, therefore, as the more general term and any reference to the standard bar may be understood to signify the standard gauge line on a bar separate from the structure.

Reference Length and Reference Observation: In order to determine changes in length of instrument it is necessary to make comparison of all observations on the standard gauge line. To facilitate this comparison the length of the instrument at the time of some reference observation on the standard may be chosen as a reference length. A comparison of all other readings on the same standard gauge line with the reference observation chosen will show whatever variation there is in the length of the instrument. For convenience the first observation on the standard gauge line has been assumed as the reference observation. The length of the instrument at the time of this observation will then be known as the reference length.

7. *General Outline of Method of Testing.*—After determining what measurements will best give the information desired from the test, the gauge lines are laid off on the surface of the concrete and small holes are cut or drilled in the concrete at a predetermined distance apart in order to expose the steel or allow a metal plug to be inserted, according as the measurement is of steel deformation or concrete deformation. The metal plugs used are securely held in place by embedment in plaster of paris. The gauge holes having been carefully prepared, a set of zero observations is taken on all gauge lines, an increment of the loading material is then applied and a second series of observations on the gauge lines is taken. The difference between the two observations on the same gauge line represents the deformation in that gauge line. It is possible that this apparent deformation may be due partly to temperature changes in the instrument instead of stress changes of the material by reason of applied load. For this reason reference measurements are made on standard unstressed bars made of invar steel which has a very low coefficient of expansion and whose change in length due to change in temperature would therefore be very slight. From these readings on the standard bar, temperature corrections are computed as shown in a later paragraph and applied to the observations in order to determine the actual change in length of the gauge line. Another increment of load is then applied and another series of observations taken.

Floor deflections also have been measured in all of these tests, but they have been considered as of secondary importance. They have been used to throw light on the correctness or incorrectness of the deformation readings and to gain some idea of the general distribution of stresses throughout a floor. Apparently they can be depended upon to show with considerable accuracy the proportional rate of increase of stress, but deflection formulas are so imperfect that measurements of deflections can not be depended upon to give actual values of stresses.

Measurements of dimensions such as span, depth of beams, location of

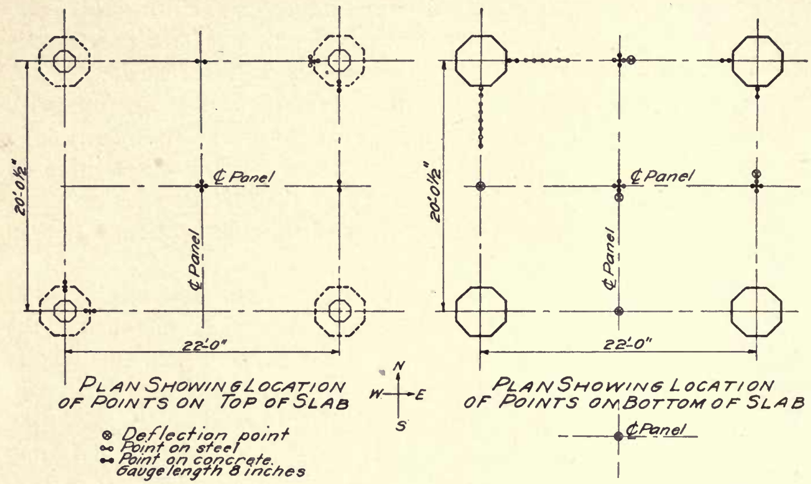

FIG. 2. CARLETON BUILDING TEST; PLAN SHOWING POSITION OF GAUGE LINES.

observation points, weight of loading material, location of cracks and any other measurements which are of value in working up results are carefully taken.

The gauge lines are usually distributed over and under the surface of the floor tested in order to gain an idea of the changes occurring in different parts of the structure. The statements in the preceding paragraphs give in general terms the features of a field test. There are many difficulties to be overcome and many chances for error. The methods of overcoming the difficulties and avoiding the errors will be discussed in the following pages. Most of the statements there made represent the results of experience on building tests.

8. *The Planning of a Test.*—Each test made will involve individual consideration of the choice of area to be loaded, the number and location of gauge lines and deflection points, the number of laborers required, the loading material to be used and its distribution, and the provisions for storage of the loading material near the test area without appreciably affecting the stresses which are to be measured. Other matters will come up for consideration but generally different solutions will be required for each test.

The area to be loaded should be chosen so as to fulfill the following conditions as completely as possible.

(a) It should be so located as to give conditions in the beams, slabs, columns, etc., as severe as will be found anywhere in the building when in use.

(b) It should be free from irregularities of construction.

(c) It should be as free as possible from disturbances by workmen.

(d) It should be as easily accessible to the loading material as possible.

In most cases some limitation is found on part or all of the conditions named. For example, in the test of the Wenalden Building it was impossible to find an area entirely free from irregularities of construction. An industrial track crossed one of the panels chosen, and the floor was thicker immediately under this track than at other places. On the edge of one or two of the panels tested, beams about an inch deeper than the regular beams were located. However, none of the measurements assumed to give typical results were taken in these panels, and it is believed that the stresses in the other panels were not affected appreciably by the irregularities. Again, in the test of the Franks Building it was not possible to choose a lower floor convenient to the loading material. The choice of an upper floor fulfilled one of the conditions mentioned—it gave a much more severe test of the columns than a test on a lower floor would have done. Also, in the test of the Carleton Building at St. Louis the area to be tested was specified by the city building department, and there was no choice as to location, on the part of those making the test.

The number of measurements to be taken will depend upon the nature of the test, the number of observers, and the number of laborers. If the test is a part of a series by which it is expected to gain scientific information which will afford a basis for design, it is likely that it will be made deliberately enough that a large number of measurements may be taken. Such tests were those of the Wenalden Building, the Franks Building, the Turner-Carter Building and the Barr test panel. If, on the other hand, the test has more of a commercial nature or is a utilization of the opportunity offered by the acceptance test to take some measurements which will show actual stresses, or if for any other reason the test is hurried, the number of measurements will necessarily be rather small. Of this class, the tests of the Carleton Building in St. Louis and of the Ford Motor Building in Detroit, Michigan, are good examples. Notice was given the engineers only about one day in advance that a test would be made on the Carleton Building. Permission was obtained from the contractor to expose bars for measurement in various places and to erect the necessary scaffolding. The measurements were made more for the purpose of checking the analysis upon which the design was based than to form in itself a basis of design. Therefore comparatively few observation points were used. It is believed that this test is representative of the type of test which is practicable on a commercial basis, hence

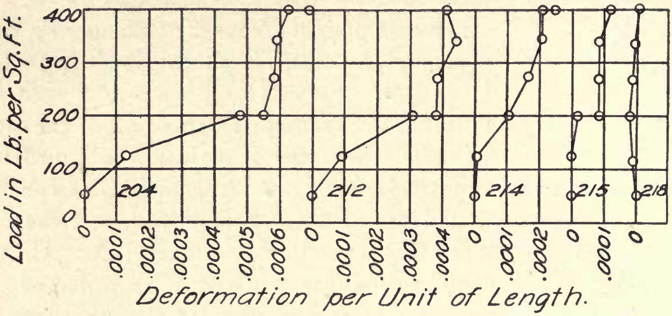

FIG. 3. POWERS BUILDING TEST; LOAD-DEFORMATION DIAGRAMS FOR SERIES OF GAUGE LINES ON REINFORCING BAR.

(by courtesy of the Corrugated Bar Company) a plan is given in Fig. 2 showing the points where measurements were taken.

The principal subjects for investigation in any test will determine the arrangement of observation points. Whatever the subject of study may be, the observation points should be arranged in such a way that a curve of deformations may be plotted against distance, showing a gradual progression from the condition at one part of the structure to the condition at another, for it is found that even under the most careful work there are inconsistencies which will make the results look doubtful if standing by themselves. The points so arranged should be numerous near the place where the measurements of greatest importance are to be taken, so that the results will not depend upon measurements at a single point, or upon the average at portions of the structure supposed to be similarly situated but in different parts of the building where unknown conditions actually may cause a large variation in the phenomena of the test. It will not be possible to carry out this plan for all subjects of investigation, as the number of observations required usually would be impracticably large. Such provisions may be made to cover the main lines of investigation, and isolated observation points may be used to gain information as to tendencies of other portions of the structure, but of course less reliance must be placed on the results of the latter measurements than where the larger number of observations is made. It would be advantageous, as was done in the Powers Building test and also in the Barr panel test, for two observers to check measurements on the same points. One or both of these checks is very valuable in establishing the correctness of observations. Fig. 3, 4, 5, and 58 illustrate the former method. Fig. 3 gives the load-deformation diagrams for a series of gauge lines in the test of

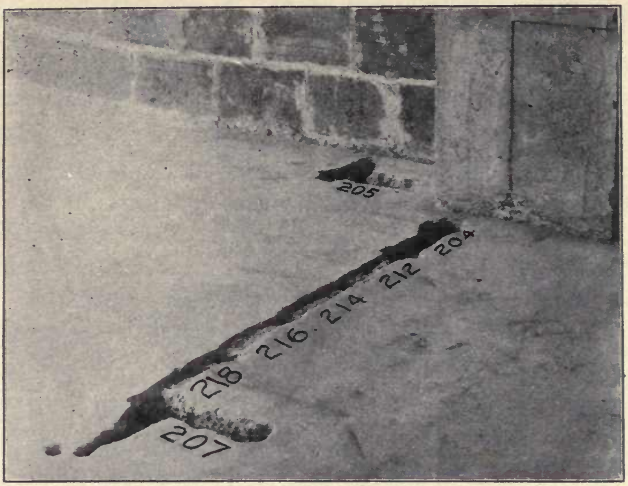

FIG. 4. POWERS BUILDING TEST; LOCATION OF SERIES OF GAUGE LINES.

the Powers Building. Fig. 4 shows the location of these gauge lines with reference to the wall and a column. Fig. 5 shows the same data plotted as deformation against distance from the column instead of against load. It may be seen that the correctness of the load-deformation curve for one of these points, if standing by itself, might be doubted because of the complete change in the character of the curve at a load of 200 lb. per sq. ft. But when these deformations are plotted against distance, the results look so consistent that it is scarcely conceivable that they are seriously incorrect. In the test of the Wenalden Building very high compression deformations were observed in the beams near the sup-

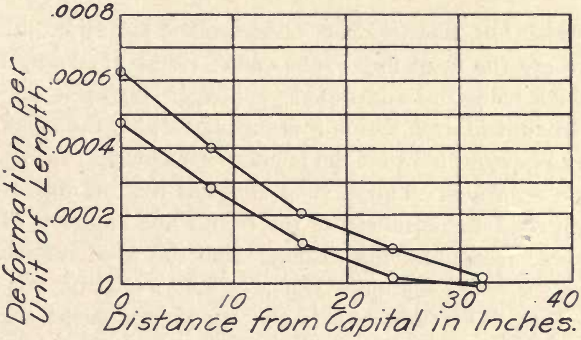

FIG. 5. POWERS BUILDING TEST; DATA OF FIG. 3 PLOTTED AS DISTANCE-DEFORMATION CURVES.

ports. As this was the first test of a beam and girder floor it was considered especially important that in the next test, that on the Turner-Carter Building, evidence be obtained which would give additional information bearing on this high compression in the concrete; accordingly the method of placing observation points at frequent and regular intervals along the ends of the beams was used. The deformations measured are plotted in Fig. 58, p. 80, against the distance from the supporting column, and the results not only tend to show the correctness of these measurements but also to indicate that the high stresses observed in the beams of the Wenalden Building were actually present.

The subjects for investigation will vary largely in different tests. In the tests discussed in this bulletin deformations have been measured with a view to obtaining information on each of the following subjects:

(a) The values of the coefficients of bending moment at the center of the span and at the support of the beam or slab under investigation.

(b) Relative moments at support for various conditions of fixedness.

(c) The extent to which the floor slab acts as a compression flange of the floor beam to produce T-beam action.

(d) Bond stresses.

(e) Diagonal tension.

(f) Stresses in columns.

(g) Time effect under constant load.

(h) The lateral distribution of stress to parts of the structure entirely outside of the loaded area.

(i) The extent to which steel stresses are modified by errors in the assumption that no tension is carried by concrete.

(j) Stresses in slabs of beam-and-girder floors.

(k) Relative stresses in short and long directions of rectangular panels.

Other subjects of investigation have received attention but those mentioned above are the most important ones. Some phenomena have been observed which bring out additional problems. Of these the determination of the amount of arch action present is probably the most important.

It is not to be expected that the moment coefficients can be determined with absolute accuracy. The method used has been to measure deformations on both steel and concrete at the center and supports of the beams and from these measurements to determine the total resisting moment developed at the given section. Equating this resisting moment to the bending moment kWl (where k is the bending moment coefficient), a solution is made for the value of k. Arch action, tension in the concrete and the sharing of bending moment by adjacent beams complicate the problem. It is suggested that the amount of arch action may be determined in any case by making a special study of the deformations in a cross-section at the mid-span of each beam and on the floor slab across an entire panel. In this study, deformations should be observed on the steel and at various elevations on the concrete so that the position of the neutral axis and of the center of gravity of tensile and compressive stresses respectively may be accurately determined. By this means it should be possible to determine if the sum of the compressive stresses is in excess of that of the tensile stresses. If so, the difference apparently must be the direct thrust due to arch action. A second section may profitably be taken at or near the point of inflection. The same study can be made, though not so satisfactorily, at the ends of the beams. The measurements for thrust will require observations on an extremely large number of gauge lines, and the observations must be extended far enough into the adjacent panels to determine the extent to which they share in carrying the load.

9. *Preparation for the Test.*—In all of these tests it was necessary to cut holes in the concrete in order to expose the steel. Fig. 4 shows holes cut in the concrete of the Powers Building where a series of measurements was taken on a rod which passed through a column. This cutting has been best accomplished by the use of a cold chisel with a very gradually tapering point. This work is a task for common laborers and a long one for inexperienced men. It has been found that a great deal of speed can be developed by practice, hence the importance of completing this part of the work with a single set of workmen.

A saving in mutilation of floors often can be effected by planning the test ahead of time and inserting plugs in the concrete during construction in the proper position for the gauge lines. Removal of the plugs after the concrete has set exposes the steel without the use of a cold chisel. Likewise metal plugs may be set in the concrete at the proper positions for the measurement of concrete stresses and thus save cutting into the concrete to place compression plugs. The point has been raised that by preparation of this kind a chance is given to the contractor to know what panels are to be tested and thus to make the construction of that panel better than others. For this reason there is room for question as to the advisability of using this method. Its employment will depend largely on the purpose of the test and on the conditions under which it is made. In most of the tests under consideration this point has been taken care of by the fact that it was not known until shortly before the test what area was to be loaded.

Drilling of the gauge holes will be discussed in article 14.

An elevated platform which will enable the observer to get close enough to the floor above to take observations of deflection and deformation is necessary. This should be supported at such a height that when the observer stands upon it the points where measurements of deformation are to be taken will be about one inch above his head. For flat slab construction this condition is easily obtained (see Fig. 6),

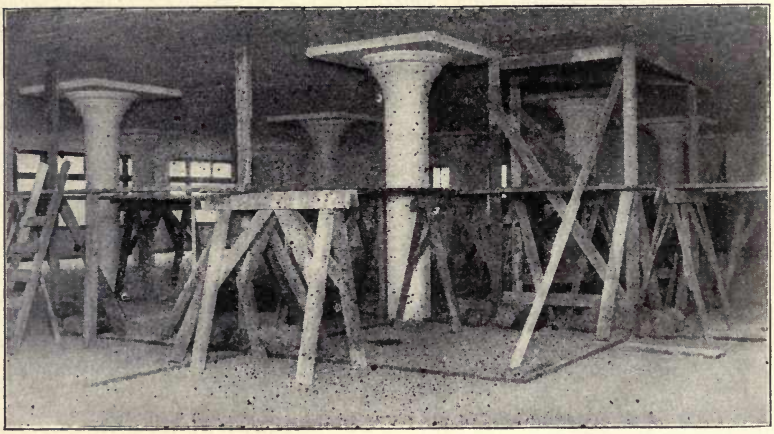

Fig. 6. Franks Building Test; Observation Platform and Deflection Framework.

but with beam and girder construction where there are measurements on beams, girders and the floor slab, the heights of different gauge lines vary so much that arrangement will need to be made for building certain parts of the platform higher than others (see Fig. 7). It is important that the elevation of the platform should be such that the observer can stand erect while taking the readings, and yet such that the instrument will not be too high for convenient and accurate observation.

Another framework for the purpose of supporting deflection apparatus under the points where measurements of deflection are to be taken is also necessary. In order that the movements of the observers upon the observation platform may not jar the deflection apparatus, the two frameworks must be built independently of each other. In all the tests which have been made, these deflection frames have stood on the floor and have been braced from one to the other in order to make a comparatively rigid framework. Fig. 6 shows scaffolding and deflection frames for the Franks test.

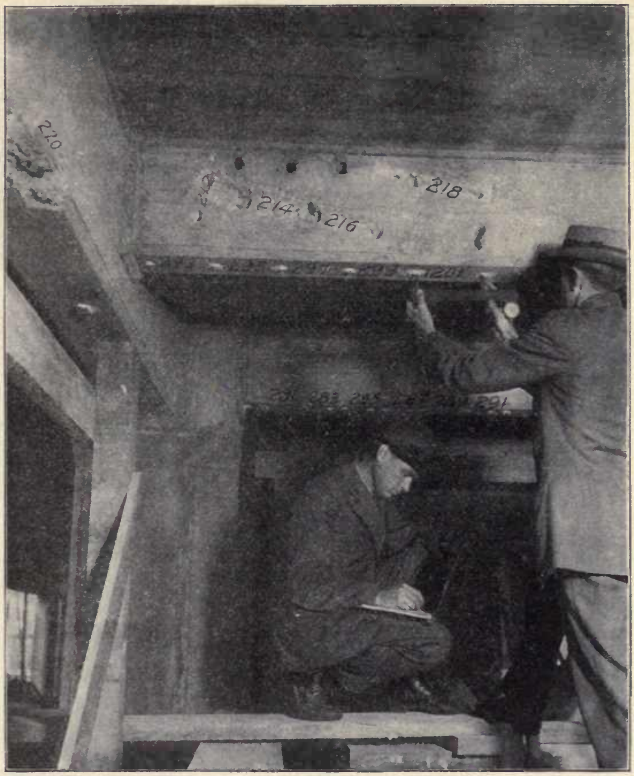

FIG. 7. TURNER-CARTER BUILDING TEST; PHOTOGRAPH SHOWING VARIATION IN HEIGHT OF GAUGE LINES.

The equipment necessarily will consist of the following: cutting and drilling tools, portable lamps for throwing light into observation holes, note books and facilities for doing drafting and for reducing data.

The cutting and drilling tools are sufficiently described in other paragraphs.

Some kind of a portable light is a necessity as gauge lines are often located in dark corners and as observations may be taken at any hour of the day or night. The lamp shown in the photograph of Fig. 16, p. 29, is a hunter's acetylene lamp and is quite satisfactory. The lamp is attached to the forehead and light may be thrown in various directions according to the setting of the clamp attachment. The acetylene tank may be attached to the belt or carried in the pocket.

Loose leaf note books should be provided in which the sheets are as large as conveniently can be handled and filed. The data sheets shown in Fig. 26, p. 44, are very conveniently ruled in hectograph ink and copied by means of a hectograph. Printed forms might be used, depending on whether the number required would justify the expense of having them printed.

For the most efficient work in computing results and making sketches for records, it is important that an adequate place be provided where quiet may be had, where benches and drafting tables may be used and where instruments and other equipment may be kept. The photograph in Fig. 8 shows the temporary office which was provided in the

FIG. 8. TURNER-CARTER BUILDING TEST; INTERIOR OF OFFICE.

Turner-Carter Building test. This is one of the portable office shanties which the company moves to places where work is being done. The photograph shows the interior of the office with the observers and recorders at work reducing the data of the test. This added equipment will add only slightly to the cost of the test but very greatly to the efficiency of the work. Special attention is called to it because the office is an important piece of equipment and it has not always been provided.

10. *Loading.*—In the tests which already have been made, the following loading materials have been used: brick, cement in bags, loose sand in boxes or bins, sand in sacks and pig iron. The material used will almost always be that which is most easily available, because the moving of loading material from any distance adds very greatly to the cost of the test. Leaving consideration of cost out of the question, sand

in sacks seems to be the most satisfactory of the materials above mentioned for loading purposes. Some of the qualities of the materials mentioned are as follows:

(a) Brick: Brick spalls and chips in handling, covering the floor with dust and jagged particles which cause discomfort to the observer in kneeling to take observations. It is important to avoid this because discomfort necessarily decreases the accuracy of his observations. This might be avoided by sweeping, but in sweeping it is difficult to avoid getting dirt into holes where observations are to be taken, and this is just as troublesome as having the dirt on the floor. Fig. 31, p. 50, a photograph of the Wenalden test, shows the use of both brick and cement in the same test. Attention is called to the proximity of the cement sacks to the beams and girders of the floor above. In some cases if cement and brick were used the intensity of the load would be limited by the height of the ceiling.

(b) Cement: Cement sifts through the sacks and the sacks become untied, scattering cement on the floor, filling observation holes and causing much dust in sweeping or cleaning up. The dust is injurious to delicate instruments and annoying to observers and recorders.

(c) Loose Sand in Small Boxes: As sand is usually damp, it does not have the fault of causing dust, and consequently is more easily cleaned up than the other materials mentioned. There are, however, other objections to it. In filling boxes it is difficult to avoid spilling the sand around and between the boxes, and consequently filling the observation holes. On account of the great difficulty in removing loose sand without spilling a great deal of it, it is impracticable to take observations as the load is being removed, therefore it is necessary to remove in one increment the whole load from a given panel. Fig. 46, p. 68, is a photograph of the Turner-Carter test and shows this method of loading.

(d) Sand in Sacks: Sand in sacks constitutes a very satisfactory loading material. An example of its use is shown in Fig. 9, a photograph of the test of the Barr Building test panel. It was piled up to a height of about nine or ten feet, and very little inconvenience was caused by the sacks coming untied or by spilling the sand. The worst difficulty encountered, and this exists with all materials handled in sacks, is that of the sliding of sacks on themselves when the load is piled high. It can be seen in Fig. 9 that bracing was necessary to prevent the sand from sliding together and filling up the aisles. It is a source of danger to those taking observations as, if a slide should occur, it probably would give very little warning and might catch the observer while in such a position that he could not escape. However, this objection would hold

FIG. 9. BARR PANEL TEST; SAND IN SACKS AS A LOADING MATERIAL.

with any material when piled as high as was that in this test. Under any circumstances it is necessary that care be taken and undue risks avoided.

(e) Pig Iron: Pig iron was used as loading material in the test of the Franks Building (see Fig. 10). From the standpoint of the making of the test, the worst objection to it is that, as with the brick, small particles break off and cause annoyance to observers. This is

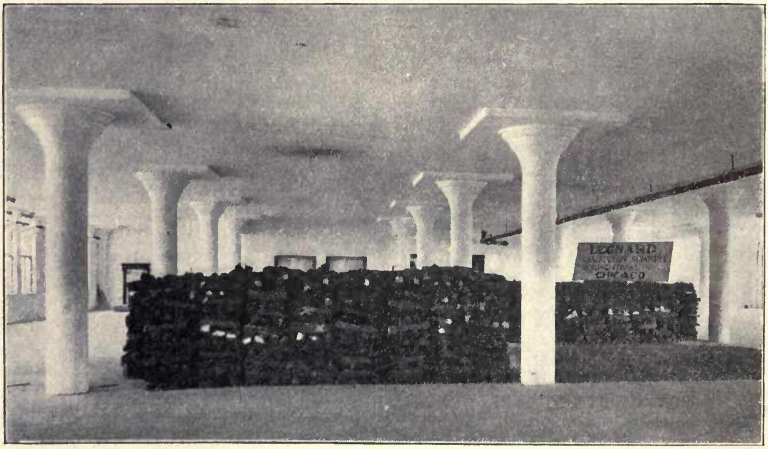

FIG. 10. FRANKS BUILDING TEST; PIG-IRON AS A LOADING MATERIAL.

less troublesome than with brick and in other ways pig iron is clean. It possesses the great advantage that with its use a very heavy load can be applied without piling the load extremely high.

Tin plate in boxes two feet square, each weighing 200 pounds, was to have been used in a building test. A more nearly ideal loading material would probably be hard to find, but unfortunately this test could not be carried out.

The intensity of the loading will depend mainly on the load for which the building was designed. It will not be possible to make the load absolutely uniform, as aisles usually will be necessary for the purposes of (a) convenience in placing the load, (b) access to gauge lines for the taking of observations, and (c) the prevention of arching in the loading materials. It has been found that it is difficult to cover more than about 75% of the actual area of the floor, and in many cases less than this will be covered. Hence in computing the probable height of the load this fact must be taken into consideration.

Aisles should be so placed that the load, even though partly carried by arching of the material, will cause stresses in the floor which approximately are equal to, and always as severe as, those caused by an actual uniform load. Fig. 11 shows the moment and shear diagrams which would be obtained by loading a simple beam with a total load W distributed over the span in three different ways, as follows:

(a) Solid Line: Total load W, uniformly distributed over full span.

(b) Broken Line: Same load W distributed over one-half of span, giving aisles of equal width at center and support.

(c) Heavy Dotted Line: Same load W distributed over one-half span, half of load being carried by arch action to ends of boxes (shown here as concentrated loads, $\frac{W}{8}$), and the other half being uniformly distributed over the half span.

It will be possible in almost any test to arrange the loading material in such a way as to come within the limits outlined by the three arrangements of load assumed in the preceding illustration, and it is seen that if this is done, the presence of the aisles or of arching to the sides of the boxes or piers, while not affecting the amount of the maximum moment and the maximum shear, would tend to cause them to exist over greater portions of the span than would be the case with an equal uniform load. In this figure aisles equal to one-quarter of the span have been assumed. In no case would they be as large as this, and therefore the moment and shear diagrams actually should conform even more nearly to those for uniform load than is shown in the figure.

Arrangement should be made, if possible, to store the loading material near the test area in order to hasten the work of applying the load after the test begins. The general rule has been to allow loading material to be stored as close as one full panel length from the test area, but the intensity of the storage load has been kept down as much as possible.

The number of laborers which can be used advantageously will depend upon the distance of the point from which the loading material is

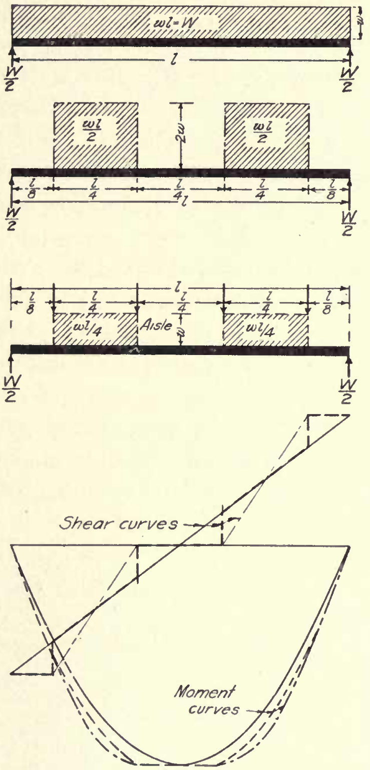

FIG. 11. MOMENT AND SHEAR CURVES FOR THREE ARRANGEMENTS OF LOAD.

to be transferred, upon the size and accessibility of the tested area, upon the amount of work which can be done by them during the intervals between increments of load while observations are being taken, and upon the length of time required to take a series of observations. The direction of the labor should not be left to the one in charge of the test, if it can be avoided, since proper attention to the conduct of the test demands all of his time. In the tests included in this bulletin the number of laborers has varied between wide limits, from 5 or 6 in the Powers test to 30 or 35 in the Deere and Webber test.

11. *Extensometers.*—In the past the great obstacle to the measurement of deformations in building tests has been the difficulty of attaching the measuring instruments to either the steel or the concrete on the flat surface of a floor, and recent tests show the necessity of making measurements of steel deformation directly on the steel. A satisfactory method of accomplishing this has been provided by the introduction of the extensometer invented by Professor H. C. Berry of the University of Pennsylvania and adapted to this work by modifications and improvements made at the University of Illinois. This instrument is similar in some respects to the strain gauge designed and used as long ago as 1888 by James E. Howard, Engineer-Physicist of the Bureau of Standards, and until recently Engineer of Tests at Watertown Arsenal.

The great value of this instrument in building tests lies in the following facts: (a) Its use makes it possible to take measurements directly upon the surface of the steel and concrete. (b) With its use there is no apparatus left in place to be damaged or disturbed during loading. (c) Due to the fact that it is portable, measurements may be taken in a large number of places with a single instrument. Measurements have been taken at as many as 268 points in a single test. If fixed instruments were used this would call for an outlay of from $2500 to $5000 for instruments and the impossibility of keeping so many instruments in adjustment under testing conditions would render their use impracticable.

Fig. 12 shows the Illinois extensometer in its present form.* Any

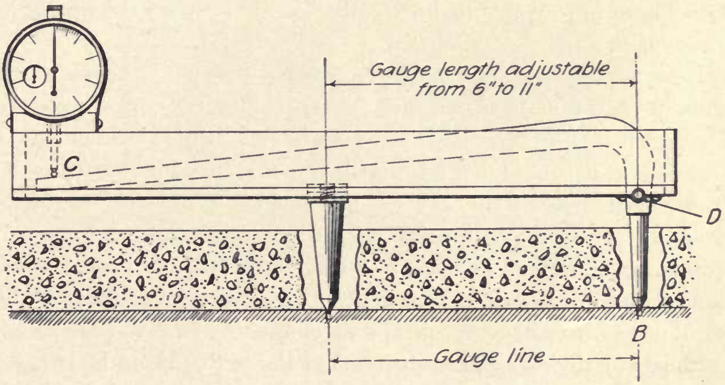

FIG. 12. ILLINOIS TYPE OF BERRY EXTENSOMETER.

*Another form has been devised since the manuscript for this bulletin was prepared and this newer extensometer is now in use in laboratory tests.

movement of the point B due to a change in the length of the gauge line is transmitted to the Ames gauge through vertical movement of point C, by means of the leg BD and the arm DC pivoted at D. The Ames gauge is sensitive to a movement at C of .0001 inch. The ratio of the length CD to the length BD is approximately five and the Ames gauge is thus sensitive to a movement at B of .00002 in. (.0001 inch ÷ 5). However, this must not be taken to mean that the extensometer possesses this degree of accuracy in measuring stresses, since some movement of the point of the leg at B is certain to result from variation in the handling of the instrument.

To obtain the exact ratio between movements at points B and C the instrument is calibrated by means of a Brown and Sharpe screw micrometer. For known movements of the point B readings of the Ames gauge are taken and a calibration curve plotted for the entire range of the instrument.

The first instrument of this type built by the Engineering Experiment Station of the University of Illinois was made for the Deere and Webber test. It was designed by Professor H. F. Moore and Mr. A. R. Lord, and was like the instrument in use at present except that it had a 15 in. gauge length and was made entirely of steel. Later on in making the instrument for general use aluminum was substituted for steel in order to reduce its weight and the gauge length was made variable from 6 in. to 11 in. Since then several minor changes have been made. The legs have been made stiffer in order to reduce the error due to unconsciously applied longitudinal thrust and the points have been made sharper in order to reduce the pressure required in seating the instrument. These improvements have considerably reduced the probable error of observation.

The extensometer loaned by Professor Berry to the University of Illinois in 1910 for use as one of the instruments in the Deere and Webber test is shown in Fig. 13. It differed from the Illinois instrument in that the movement of the multiplying arm was measured by means of a screw micrometer instead of the Ames gauge head, the point of contact of the micrometer plunger and the lever arm being determined by means of a telephone apparatus. The screw micrometer and the frame of the extensometer were insulated from each other and were connected with the poles of a small battery by means of copper wires. A contact between the plunger of the screw micrometer and the multiplying lever completed the circuit and the current set up produced a vibration of the diaphragm of the telephone apparatus carried on the head. This method of observation was very slow and the electrical connection got out of order very easily.

FIG. 13. ORIGINAL BERRY EXTENSOMETER IN USE.

The use of the Ames gauge head (instead of the screw micrometer and telephone apparatus) adopted by Professor Moore in the instrument used in the Deere and Webber test has greatly facilitated the use of the extensometer. The legs of this instrument also were made longer in order to adapt it to the measurement of deformations of steel embedded in concrete. Both of these modifications later have been used by Professor Berry in instruments which he has put upon the market.

The extensometer more recently designed by Professor Berry is shown in Fig. 14. It is not different in principle from the one just described.

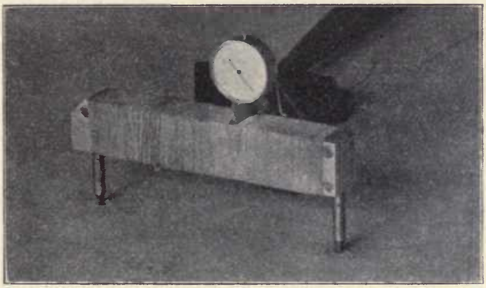

FIG. 14. NEW BERRY EXTENSOMETER.

It differs from the Illinois instrument in the following details: (a) Instead of having a uniformly variable gauge length ranging from 6 in. to 11 in., it has two fixed gauge lengths of 2 in. and 8 in. respectively. (b) The instrument shown here has a multiplication ratio of two between leg and arm, and in order to make this ratio five (as in the Illinois type) it is necessary to use a leg which is only one inch long. With this arrangement the instrument cannot usually be used for measuring deformations in reinforcing bars in place, owing to their depth of embedment. (c) The framework of this instrument is of invar steel or of aluminum. While invar steel makes the weight somewhat greater than that of the aluminum instruments, it has the advantage of reducing the dependence on an invar steel standard bar and the study of the effect of temperature changes in the steel and concrete of the structure is accomplished with greater ease.

Mr. F. J. Trelease of the Corrugated Bar Company has designed an instrument of the Berry type and has used it in at least one test. This instrument, shown in Fig. 15, also has as its main feature a multiplying

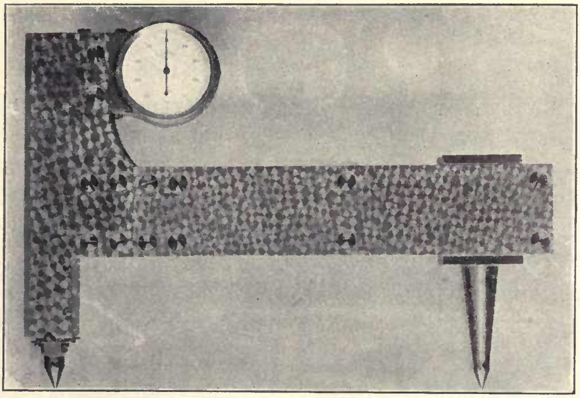

FIG. 15. EXTENSOMETER DESIGNED BY F. J. TRELEASE.

lever which actuates the plunger of an Ames gauge head. The principal difference between this instrument and the one shown in Fig. 12 is that the multiplying lever is vertical instead of horizontal. Results have been obtained with it which do not differ much as to accuracy with those of the Illinois type of instrument.

12. *Standard Bar.*—The necessity for a standard bar was first found in the test of the Deere and Webber Building. Variation in temperature was sufficient to cause a change in the length of the instrument as great in many cases as that in the reinforcing steel due to the applied load. Hence it was found necessary to make observations on an unstressed standard bar to show any temperature changes in the length of the instrument. In this test a bar of about ⅝-in. steel was used

as a standard. It was protected from rapid temperature changes by embedment in plaster of paris, and was kept on the floor where the test was being made. In this way it was expected to make the change in the length of the standard bar due to temperature variations about equal to the change in length of the reinforcing steel due to the same cause. To some extent this purpose was accomplished, but as the plaster covering was thin and not very dry, the change in the standard bar must have been much more rapid than that in the reinforcing steel. In the test of the Wenalden Building and of the Barr test panel, precautions were taken to embed a standard bar in concrete. This was done also in the tests of the Powers Building and of the Franks Building, and in addition standard gauge lines were established in parts of the floor not affected by the load. In the Turner-Carter test only the latter method was used. These standard gauge lines have been placed both on the reinforcing steel and in the concrete. Fig. 16 shows the taking of an

FIG. 16. TURNER-CARTER BUILDING TEST; TAKING AN OBSERVATION ON A STANDARD GAUGE LINE.

observation during the Turner-Carter test on a standard gauge line. The standard gauge line is located in a part of the floor entirely away from the loaded area.

The greatest development in the use of the standard gauge line has been in the frequency of reference to it and in the development of an exact system for the calculation of temperature corrections. It was noted previously that a steel extensometer was used in the Deere and Webber test but that in the subsequent tests an aluminum instrument was used. Since the coefficient of expansion for aluminum is almost twice that for steel, it is apparent that dependence on a standard must have been of still greater importance in the later tests. Difficulty was found in interpreting the notes taken on the Wenalden test, but the greater dependence on the standard gauge line and the more systematic use of it since then has very largely overcome this difficulty. In the test of the Larkin Building standard bars of invar steel were used. Invar steel has a very low coefficient of expansion and its use as a standard bar makes it possible to eliminate from the measurements almost all the effects of temperature variation in the extensometer. If it is desired to determine how great are the temperature effects, a standard gauge line can be placed in the floor as before in such a position as not to be affected by the floor load.

It has been the practice in the more recent building tests for each observer to make observations regularly on two standard gauge lines. This is done in order that one may form a check on the other. If only one standard were used, a large accidental change in the readings due for instance to sand in the gauge holes might be mistaken for a temperature effect. If two standards are used, such an accidental change as the above seldom would be the same in both, and the error would be detected. An accident to the instrument would probably cause the same change on both standard gauge lines and the use of the two standards would not help to detect this kind of an error. However, such errors are usually so large as to be apparent in a reading of the standard gauge line and are infrequent as compared with errors due to dirt in the gauge holes.

13. *Deflection Instruments.*—In the building tests referred to in this discussion deflection instruments of two types have been used, one being that used by the Illinois Engineering Experiment Station and the other that used by the Corrugated Bar Company. The former consists of a screw micrometer head of 1-in. travel, connected in tandem with an Ames gauge head micrometer of ¾-in. travel. The screw micrometer is designed to cover large variations in deflections and the Ames gauge head small ones. The Ames gauge head shows an increase in reading for a decrease in length of the deflection instrument, and a screw micrometer head, as ordinarily constructed, would show a decrease in read-

ing for a decrease in length. Thus to obtain an observation which involves the readings of both the Ames gauge head and the screw micrometer head it is necessary to take the difference of these readings, but in making calculations a sum is much more easily and accurately handled than a difference. To permit addition, the graduation on the screw micrometer head used in this deflection instrument has been reversed so that it shows an increase for a decrease in length, just as with the Ames gauge head. Fig. 17a shows this deflection instrument and also

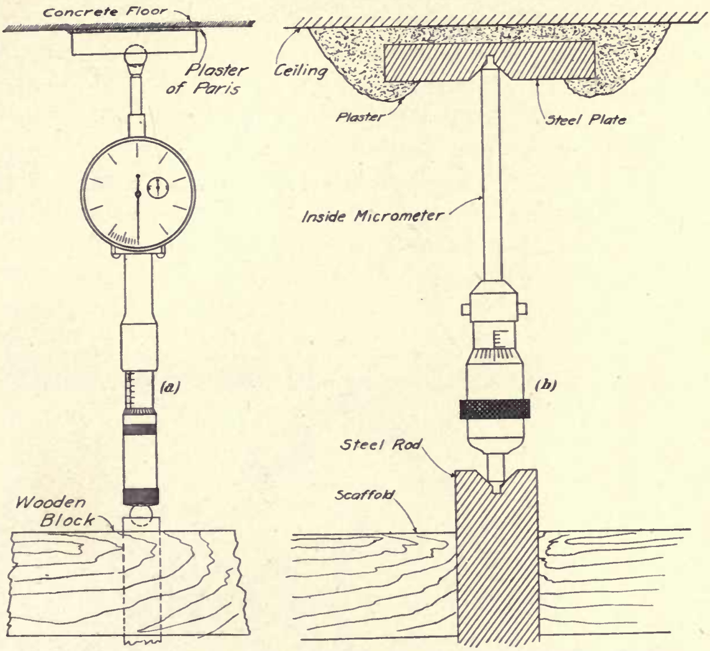

FIG. 17. (a) DEFLECTOMETER; UNIVERSITY OF ILLINOIS TYPE.
(b) DEFLECTOMETER USED IN CORRUGATED BAR COMPANY'S TESTS.

the method of using it. A plate having a ½-in. steel ball attached is plastered to the surface, deflections of which are to be measured. A ⅝-in. bolt, which has a steel ball inserted into its upper end, is set into a wooden block (part of the deflection framework) in such a way that its elevation can be adjusted to give any desired zero reading of the deflectometer. Thus at the beginning of a test all the zero deflection readings can be determined, so that for a considerable length of time all the change in deflection will be shown on the Ames gauge without any change of the screw micrometer. As larger changes take place, a second setting of the screw micrometer will probably be necessary. The great advantage of this instrument is the rapidity with which it can be used. It has been found to work very satisfactorily in most respects.

The deflection instrument used by the Corrugated Bar Company is shown in Fig. 17b and consists of a screw-micrometer depth-gauge by means of which distances for varying loads are measured between the stationary frame and a point on the beam or floor slab. It has the advantage of a much larger range of measurement than is found in the Illinois instrument. In the Barr panel test a gross deflection of more than 3 in. took place. As the Illinois type of deflectometer has a range of only 1¾ in., it could not have been used in this test. This amount of deflection, however, is more than would be likely to occur in the test of a building. The disadvantage of the Corrugated Bar Company instrument is that it requires a longer time to make an observation than does the deflectometer previously described.

14. *Extensometer Observations.*—In obtaining good results with this type of extensometer, a great deal depends upon careful manipulation. Two things which are of great importance in this respect are (a) the preparation of the gauge holes, and (b) care and experience in the use of the instrument.

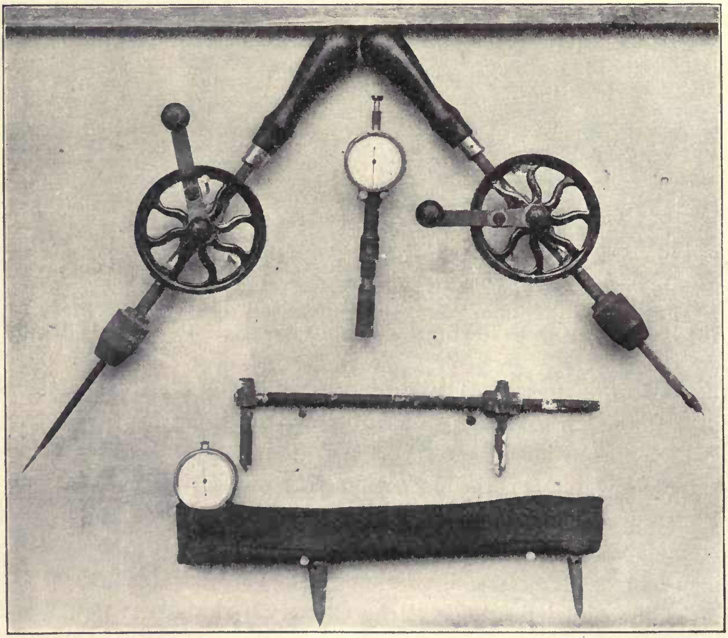

FIG. 18. TURNER-CARTER BUILDING TEST; INSTRUMENTS AND TOOLS.

The placing of the gauge holes so they will come within the range of the extensometer is best accomplished by the use of some kind of gauge marker such, for instance, as is shown in Fig. 18. In the work of the Illinois Engineering Experiment Station the holes are drilled with a No. 54 drill (.055 in. in diameter). At the beginning of the use of the Berry extensometer a number E countersink drill (approximately $^3/_{32}$ in. in diameter) was used; but a smaller one seems to be better, because it is easier to get the properly finished hole, because a slight eccentricity of the gauge holes on the reinforcing rod (see Fig. 19) causes less error in manipulation of the extensometer when a small drill is used, and because, in the case of measurements on small rods, the $^3/_{32}$-in. drill cuts away a large percentage of the steel in the rods.

A breast drill geared so that one revolution of the crank gives about 4¼ revolutions of the drill had been used previously. In the hands of a skilled workman very satisfactory work can be done in this way, but where, as quite frequently will be the case, the drilling has to be done by persons not familiar with this kind of work something better is needed. A drill driven by a flexible cable attached to a small electric motor giving a speed of rotation of 400 r.p.m. or more does better and more rapid work. Where high carbon steel is encountered fewer drills are broken, and when a hole is drilled a better job is usually the result.

After drilling the holes, the edges should be finished to remove the burr and to round off the sharp corners. The tool shown in Fig. 19 is

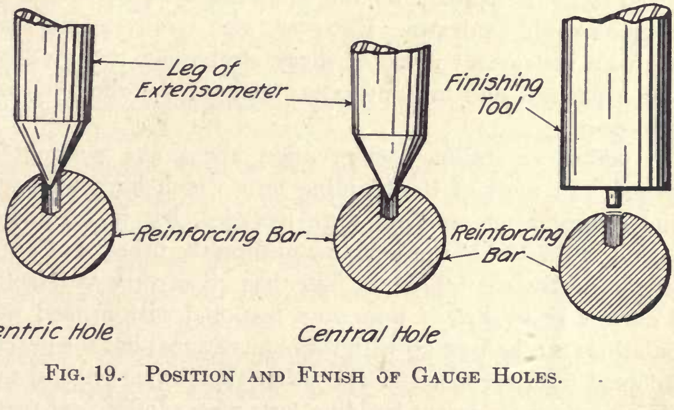

FIG. 19. POSITION AND FINISH OF GAUGE HOLES.

designed to accomplish this purpose. Such a tool should not be a cutting tool but rather a wearing or polishing tool. A pointed magnet to remove steel dust and small fragments of steel torn off in drilling, would be of use. It is hard to place too much emphasis on the proper preparation of gauge holes.

Observers should be experienced in the use of the Berry extensometer before undertaking work on a field test. The chances of error in the manipulation of the instrument are large. As a rule the deformations measured are small so that the error is likely to be quite a large proportion of the total measurement, hence it is important to reduce errors to the lowest possible limit.

If the observations at zero are as reliable as other observations, a curve may be drawn through all the points of any load-deformation diagram, weighting the zero observations equally with the others; the zero point shown by the intersection of the most probable curve should then be used as the origin. This method involves waiting until the completion of the test to draw these curves. It would be much better to repeat the observations at zero load several times and to give more care and time to their determination than is given at any other load. By this means a check can be had upon the action of the structure as the test progresses, and the construction of the most probable curve will be made more simple. To obtain a satisfactory zero point, then, it is essential that several complete series of observations should be taken with no load on the floor, and it would be well to repeat this through considerable range of temperature to study temperature effect on the steel and on the concrete. This study was attempted in the Deere and Webber test, but the changes both in instruments and in reinforcement were included in the measurements and could not be separated, so no definite conclusions could be drawn. However, with an invar steel standard bar or with an instrument made of invar steel these two sorts of changes can be separated and to some extent at least the effect of temperature determined.

In taking an ordinary observation about five readings should be averaged. In most of the building tests which have been made, individual extensometer readings were recorded, but in certain laboratory tests and in the test of the Larkin Building the practice of averaging the results mentally was followed. This has given very satisfactory results and saves a great deal of time on a test, and with a good recorder the calculations can be kept up with the observations; but the practice should be adopted with caution and only after some experience in this kind of work. In the more recent building tests where individual readings were recorded, the practice followed in making an observation has been to reject all readings until five consecutive readings have been obtained which agree within .0004 in. These five consecutive readings then are averaged to form an observation.

Deflectometer observations have been sufficiently discussed in the description of the deflectometer and will not be considered here again.

15. *Observation of Cracks.*—Up to very recently the observation of cracks has been considered one of the most important features of a test. Although it is not considered so important as formerly when strains were not measured, if carefully done it may form a valuable check on the measured deformations. These observations should be made and recorded for zero load and at each increment of load. This is one of the most tedious parts of the test, and to carry it out faithfully requires a great deal of patience. The examination should be minute and very thorough. One who is not familiar with this kind of work will be likely to miss important indications, and careful supervision should be maintained over this part of the investigation.

Special attention has been called to observation of cracks because of incorrect ideas which apparently prevail with regard to them. It seems to be the opinion of some engineers that the type of construction advocated by themselves is immune from cracks. When it is remembered that plain concrete fails in tension at a unit deformation of about .0001, it is apparent that cracks must form when the stress in the steel is such as to correspond with this deformation, or at about 3000 lb. per sq. in. At this stage the cracks are often too small for detection with the unaided eye, but with care in observation almost always very fine cracks can be seen at stresses ranging between 3000 and 10000 lb. per sq. in. Thus to report for a floor loaded to twice the design load that no cracks were observed is to admit one of three things; that an excess of steel was used, or that sufficient care in taking observations was lacking, or that not all the facts of the case were reported. It should be borne in mind that the cracks referred to in this discussion are often extremely minute and usually are not visible to a casual observer. Frequently cracks have been traced with a lead pencil to make them distinct for the purpose of sketching, and it seems apparent that some persons visiting the test have mistaken these pencil marks for large cracks. At any rate reports have been circulated as to the existence of large cracks in a test where, to the writers' personal knowledge, there were none but minute cracks.

16. *Accuracy of Deformation Measurements.*—The ratio of multiplication in the Berry extensometer is not exactly equal to the ratio of the length of the arm to the length of the leg, the error being due to the fact that the plunger of the Ames gauge head does not always travel in a line perpendicular to the multiplying lever. However, calculations show that this approximation results in an error in the measurement of

steel stresses equal to only about one-quarter of one per cent for an extreme case. It may be seen later that errors of observation are so large in proportion that this error can be neglected.

In forming a basis for a conclusion as to the accuracy of the figures given out as results of tests, use has been made of the check readings taken by two observers on the same gauge lines and of calculated probable error of the mean of five readings. While it is possible to calculate with some accuracy the probable error of replacing the instrument on the same gauge line time after time at one sitting, it is very difficult to determine the error caused by gradually cramping the quarters of the observer as the loading material piles up. Consequently the probable error calculated from a number of readings taken on the same gauge line at different sittings will in general be larger than that calculated from the same number of readings if taken at a single sitting. However, as experience develops skill in replacing the instrument at a single sitting, experience will also increase the consistency of results obtained at widely different times, and the calculated probable error will be a measure of relative, but not of the actual, accuracy of observation. A determination of errors based on independent checking by a second observer should be expected to eliminate to a large extent errors of all kinds and the greatest dependence should be placed on this kind of results.

In the test of the Powers Building most of the observations taken were checked by a second observer and some of the results are shown in the load-stress curves of Fig. 20. The values shown in solid circles were observed by Mr. F. J. Trelease and those in open circles, by Mr. Slater. The zero reading for the latter is in each case at a load of 50 lb. per sq. ft., and in order to make a direct comparison of results, all these curves must be set over so that their zeros coincide with the stress values at 50 lb. per sq. ft. of Mr. Trelease's curves. Having made this correction the average variation between all the comparable points is about 670 lb. per sq. in. (.0000223 unit deformation), which amounts to a probable error of approximately ±340 lb. per sq. in. (±.000011 unit deformation).

Fig. 21 shows the results of a series of measurements taken in the same way on the upper and lower surfaces of a 4-in. by 4-in. timber beam loaded with sacks of sand on a 12-ft. span. The points in open circles represent measurements on the top surface and those in crosses on the bottom surface. Determined in the same way, these measurements show an average probable error of approximately ±.000017 unit deformation.

In Fig. 22 is given a curve which shows for each of four building tests the probable error of the average of five readings. Each plotted point is the average of the probable errors calculated for six different gauge lines at a given load. What this diagram may be expected to

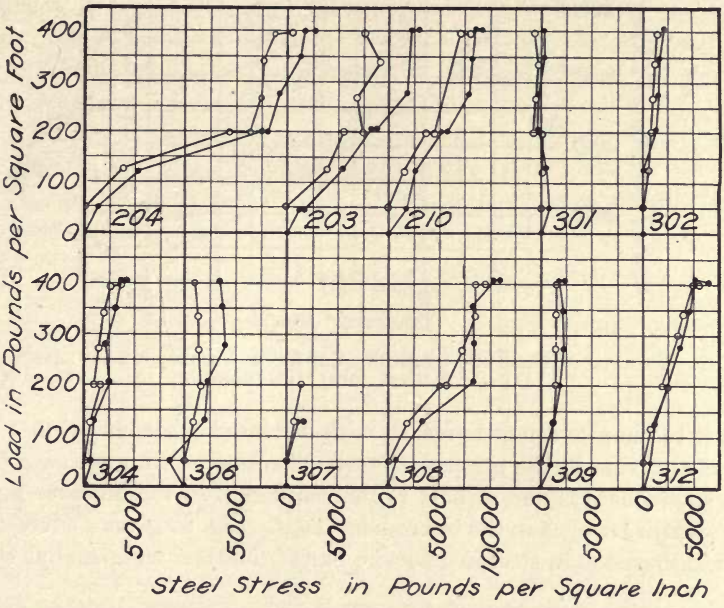

FIG. 20. POWERS BUILDING TEST; LOAD-DEFORMATION CURVES OF TWO OBSERVERS.

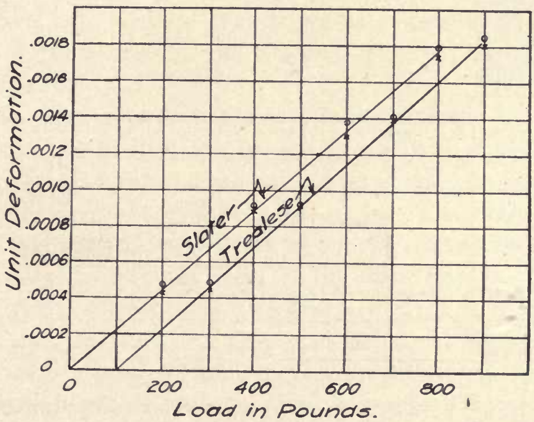

FIG. 21. 4 x 4-INCH TIMBER BEAM TEST; LOAD-DEFORMATION CURVES OF OBSERVATIONS MADE TO COMPARE INSTRUMENTS.

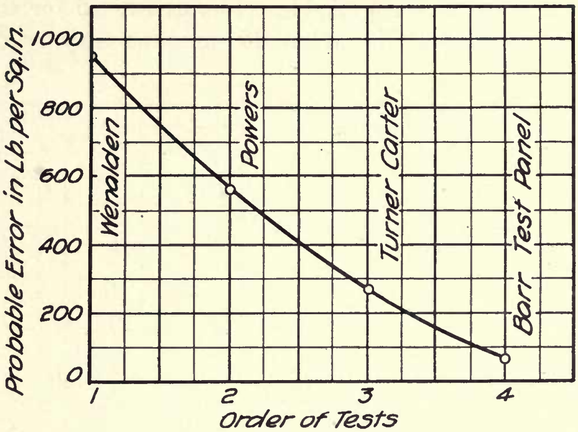

FIG. 22. PROBABLE ERROR; DIAGRAM SHOWING VALUES CALCULATED FROM DATA OF FOUR BUILDING TESTS.

show is the improvement in results with increased experience rather than the actual value of the probable error. The marked improvement in results shown here is due in part to increased skill in the observer and in part to improvement in the instrument itself. Fig. 23 gives a curve showing deformations in steel in a bottom bar of the Barr test panel as shown

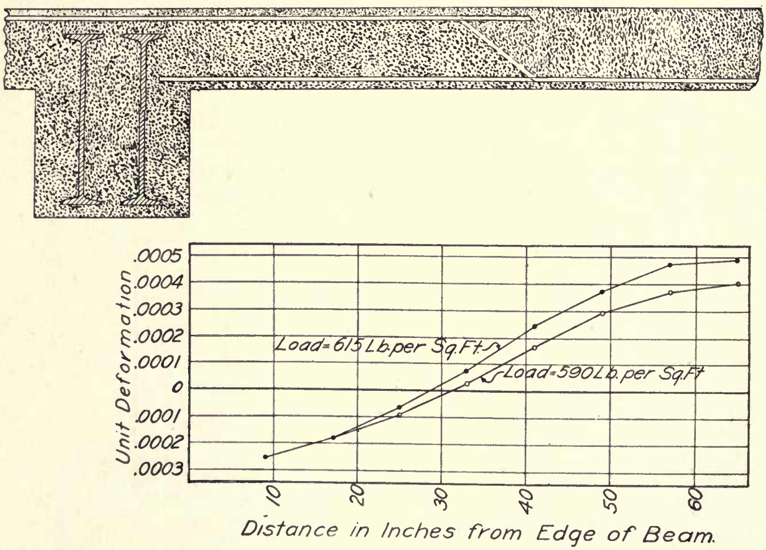

FIG. 23. BARR PANEL TEST; DIAGRAM SHOWING DEFORMATION ALONG BOTTOM REINFORCING BAR.

in the sketch. The points shown as open circles are for a load of 590 lb. per sq. ft. and solid circles are for a load of 615 lb. per sq. ft. This is the best curve the writers have been able to obtain on any building test, and it can not be taken as representative, but rather to illustrate what may be obtained under the best conditions. The regularly varying differences for a small difference of loads indicate that the stresses must have been determined correctly within a very small range.

A study of probable error was made in the Turner-Carter test by the use of a series of 100 observations taken by each of the two observers on two gauge lines selected as likely to give the most and the least accurate results. The results of this study are given in Table 1.

TABLE 1.

Probable Error of the Average of any Group of Five Consecutive Readings.

	Observer	Gauge Line		
		1	2	Average
Unit deformation	H. F. Moore W. A. Slater	.00000687 .0000043	.0000106 .000014	.00000873 .0000091
Stress in steel in lbs. per sq. in.	H. F. Moore W. A. Slater	206 130	318 435	262 282

While these measurements were not all on steel, the probable error has been reduced to terms of stress in steel for convenience of interpretation. It is very interesting to note that the average probable error of ±282 lb. per sq. in. agrees very well with that for the Turner-Carter test as shown in the curve of Fig. 22. The same observer took the data in both cases, but the data for the value shown in Fig. 22 are taken directly from the records of the test and represent the conditions on six typical gauge lines.

From the data in hand it seems safe to conclude that under difficult conditions stresses in steel can be determined with ±500 lb. per sq. in. and that under favorable conditions with careful work it may be determined within ±200 or ±100 lb. per sq. in. The advantage of further increase in accuracy of results lies in the determination of the relations between stresses in different parts of the structure.

17. *Effect of Changes in Temperature on Accuracy of Results.*—Changes of temperature will give measurable changes of length in reinforcing steel, in concrete and in instruments made of ordinary materials. In most of the building tests, corrections have been made for the changes in the instrument due to changes in temperature by means of observations on standard unstressed gauge lines chosen to represent

as nearly as possible the conditions of the steel and the concrete in the part of the structure tested. The method of calculating this correction will be described in a later paragraph. It is there mentioned that in distributing the corrections found by reference to the standard bar, a linear variation from the time of one standard observation to the time of the next standard observation was assumed. Some observations have been made to determine the correctness of this assumption.

To determine the amount of change in length of an aluminum extensometer covered and uncovered, a test was made in which the two instruments were suddenly exposed to a change of temperature of 60° F. A covering which consisted of a double layer of rather heavy felt protected one of the instruments from a sudden change in temperature. The other instrument was entirely unprotected. The results of this test are shown in Fig. 24 with the change of length of the instrument plotted as ordinates against time as abscissas.

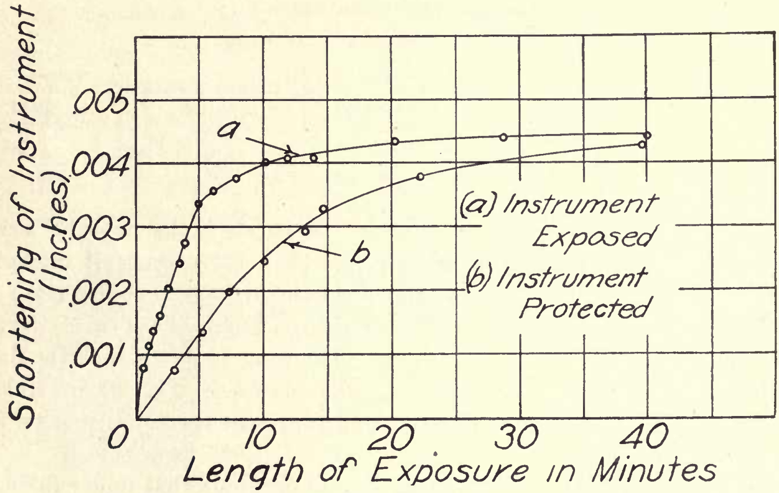

Fig. 24. Diagram Showing Change in Length of Instruments Due to Change in Temperature.

For these measurements a standard bar of invar steel was used. The coefficient of expansion of this being very small, the change of length measured must have been almost entirely that in the instrument. The curve shows that for an instrument not insulated from temperature changes, only about five minutes is required for the instrument to come to the temperature of the air. For the insulated instrument about 20 minutes was required. This may be interpreted to mean that if an unprotected instrument is used, readings on the standard bar should not be more than five minutes apart.

With an instrument protected as was this one, intervals of 20 minutes would not be too much. The amount of change for the case shown here is extreme, as the instrument was suddenly exposed to a change in temperature of about 60° F. This change would seldom be found, and the length of time required to make the change for a smaller difference of temperature may be less, but probably would not vary much for other ranges of temperature. It may be concluded that the method used for distributing the correction is justifiable, since the instrument was protected from sudden change of temperature and the observations on standard bars were usually at intervals not greater than 20 minutes.

The above test shows the effect on the instrument of change in temperature. Another test was made to determine the effect of change in temperature on steel embedded in concrete and on steel exposed to the air. A 3/8-in. square bar of steel entirely unprotected from temperature changes and a 3/8 in. round bar embedded in 1 in. of concrete were exposed to a sudden change of temperature of about 43° F. Measurements were taken on a 6-in. gauge length of each bar at very short intervals of time. The results are shown in Fig. 25. The results of this single

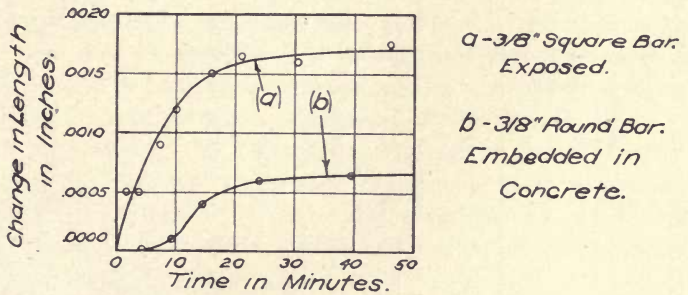

FIG. 25. DIAGRAM SHOWING CHANGE IN LENGTH OF STEEL BAR DUE TO CHANGE IN TEMPERATURE.

test must be used with caution as the total measured change in length was very small and a small error would show up very plainly. However, the curve for the embedded bar agrees in its general characteristics with some of the results obtained by Professor Woolson on "Effect of Heat on Concrete" reported in the 1907 Proceedings of the American Society for Testing Materials. The test indicates that for this range of temperature rather rapid changes may be found in the steel, corresponding to stresses of about 9000 lb. per sq. in. and 3000 lb. per sq. in., respectively, for exposed steel and steel protected as in this case. The range of temperature is extreme and the size of bars smaller than is

TABLE 2.

FORM SHOWING METHOD OF REDUCING DEFORMATION DATA.

Load	Interval	0		1	2	n−2	n−1	n	
	Gauge Line	Standards		Gauge Line Numbers				Standards	
		a	b	101	104	108	116	a	b
Dead Load Only	Uncorrected Average	S_a	S_b	R_1	R_2	R_{n-2}	R_{n-1}	S'_a	S'_b
	Correction	0	0	$\frac{1}{n}\frac{C'_a+C'_b}{2}=C_1$	$\frac{2}{n}\frac{C'_a+C'_b}{2}=C_2$	$\frac{n-2}{n}\frac{C'_a+C'_b}{2}=C_{n-2}$	$\frac{n-1}{n}\frac{C'_a+C'_b}{2}=C_{n-1}$	$S_a-S'_a=C'_a$	$S_b-S'_b=C'_b$
	Corrected Zero Average			$R_1+C_1=A_1$	$R_2+C_2=A_2$	$R_{n-2}+C_{n-2}=A_{n-2}$	$R_{n-1}+C_{n-1}=A_{n-1}$		
Any Live Load P	Uncorrected Average	s_a	s_b	r_1	r_2	r_{n-2}	r_{n-1}	s'_a	s'_b
	Uncorrected Difference			$A_1-r_1=d_1$	$A_2-r_2=d_2$	$A_{n-2}-r_{n-2}=d_{n-2}$	$A_{n-1}-r_{n-1}=d_{n-1}$		
	Correction	$S_a-s_a=c_a$*	$S_b-s_b=c_b$*	$c_{ab}+\frac{1}{n}(c'_{ab}-c_{ab})=c_1$	$c_{ab}+\frac{2}{n}(c'_{ab}-c_{ab})=c_2$	$c_{ab}+\frac{n-2}{n}(c'_{ab}-c_{ab})=c_{n-2}$	$c_{ab}+\frac{n-1}{n}(c'_{ab}-c_{ab})=c_{n-1}$	$S_a-s'_a=c'_a$†	$S_b-s'_b=c'_b$†
	Corrected Difference			$d_1-c_1=e_1$	$d_2-c_2=e_2$	$d_{n-2}-c_{n-2}=e_{n-2}$	$d_{n-1}-c_{n-1}=e_{n-1}$		

* Put $\frac{c_a+c_b}{2}=c_{ab}$

† Put $\frac{c'_a+c'_b}{2}=c'_{ab}$

often found in floor construction; therefore the results found in tests would probably be less extreme. However, this indicates the necessity of attempting to eliminate from the results of the test the effect of temperature changes, especially if the stresses measured are small.

18. *Records and Calculations.*—Since the beginning of the use of the Berry extensometer for testing purposes, as much development has been made in the keeping of notes as in the use of the instrument. Because of a lack of completeness of notes the advantages of the use of the standard bar were not fully realized for some time. Only after the method of keeping notes had been highly systematized was it possible to make properly the corrections which observations on the standard bars indicated should be made. During the time of placing an increment of load the recorder will have considerable time in which to be working up results of the series of observations taken at the previous increment of load, and as the method of making these calculations is quite intricate, a man is required for this work who has ability to do more than merely record.

It is very important on account of the great number of observations taken (about 12000 in the Turner-Carter test) that all records be arranged systematically. The following points are mentioned as being important in this connection: (1) In the field test individual readings should be recorded and their average used as a single observation unless it appears that the proposed abridgment of this procedure (see page 35) may be used safely. (2) Recording readings in the order of their size will assist the recorder in obtaining the correct readings and in rapidly obtaining the average. (3) The exact sequence of observations should be maintained in the record as the calculations of corrections depends largely on this.

Fig. 26 is a sheet for the record of original readings and the results calculated from them.

Calculating corrections and applying them to the results make the reduction of data rather intricate. This work has been reduced to a definite system indicated by the form shown in Table 2. In this system the first observation on the standard bar is used as the reference observation (see definition, p. 10). The corrections are distributed among the gauge lines as though the change in the length were a linear function of the time from one standard bar observation to the next one. These assumptions do not entirely accord with the facts but have been found satisfactory as a working basis. Any other standard bar observation than the first one would do as well for a reference observation except for matters of convenience. It is important that calculations should be kept up as the work progresses, because it can be done with less labor then than at any other time and because it will be of value to know as the test progresses what results are being secured.

19. *Test Data.*—Table 3 gives general data of the tests referred to in this discussion. The figures giving area of test show the total area

Recorder	Time	Temp.	Load	Series	TESTS OF						BUILDING,			Observer						Sheet	
					Gauge Line	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
					Corr. Zero Av.																
					Readings																
					Uncorr. Av.																
					Uncorr. Diff.																
					Correction																
					Corr. Diff.																
					Stress																
					Readings																
					Uncorr. Av.																
					Uncorr. Diff.																
					Correction																
					Corr. Diff.																
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					Readings																
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					Uncorr. Diff.																
					Correction																
					Corr. Diff.																
					Stress																

FIG. 26. FORM FOR RECORDS OF ORIGINAL AND CALCULATED NOTES.

of the floor covered, and do not count any area twice, even though loaded twice as was done in the Wenalden Building test. They do include the area of separate single panel tests which were made in the Wenalden and Franks tests.

The maximum test load in lb. per sq. ft. is given in the column under that caption. In some cases this was over only a part of the test area. The part upon which the maximum load was applied bore the following ratios to the total test area: Wenalden 80 per cent, Powers 50 per cent, Franks 40 per cent, Larkin 40 per cent, all others 100 per cent.

The column giving the amount of load handled includes the rehandling due to change of position of loads. The proportionate parts of the loads rehandled in this way were: Wenalden 40 per cent, Powers 50 per cent, Franks 80 per cent, Larkin 73 per cent. In all the other tests no load was rehandled.

The column giving the number of observers includes only those reading deformations. In the Wenalden, Powers and Larkin tests another

TABLE 3.

General Data of Tests

Building	Type	Test Area		Loading Material	Design Load lb. per sq. ft.	Test Load lb. per sq. ft.	Load Handled tons	Number of Gauge Lines	Number of Observers	Days Required for	
		In sq. ft.	Per cent of Total Area							Preparation	Test
Deere and Webber	Flat Slab	2850	14.2	Brick and Cement	225	350	500	26	3	7	5
Wenalden	Beam and Girder	1800	3.5	Brick and Cement	250	400	420	52	3	8	7
Powers	Flat Slab	1070	40.0	Cement	200	400	174	66	2	..	..
Franks	Flat Slab	1959	16.6	Pig Iron	250	624	430	95	3	..	..
Turner-Carter	Beam and Girder	1690	14.1	Sand in Boxes	150	300	260	104	2	7	10
Barr	Reinforced Concrete and Tile	1270	100.0	Sand in Sacks	150	650	348	71	3	..	9
Carleton	Flat Slab	440	1.5	Brick	150	400	90	30	1	1	..
Larkin	Flat Slab	2450	10.0	Sand in Bins	225	618	500	268	3	5	6

observer took deflection readings. In the Powers test and the Barr test, almost all the deformation readings were taken by each of two observers, giving a larger number of gauge lines per observer than appears in the table.

20. *Cost of the Tests.*—An effort was made to learn the cost of making the building tests in which the stresses in the structure were observed, but difficulty was found in separating the items connected with the tests from those incidental to the building construction. The expense of such a test depends upon the size of the test area as well as upon the number of gauge lines used. The loading of a single panel gives little information, and this information may be misleading in regard to the maximum stresses which will be developed in such a panel when the adjacent panels also are loaded. A test of a floor system should involve the loading of as many as five panels; a greater number would be more representative of full loading. The application and removal of 600 000 lb. of load involves considerable expense, especially if this material has to be brought to the building and later taken away. This item may be from $300 to $700 depending upon the distance the loading material is conveyed. The cost of building the platforms and drilling and cutting the holes for the gauge points may be counted to be about $100. A thorough test will take a week's time of the observers and two weeks' time of the one in charge of the test even though the test itself may not run over five days. A well organized party of three observers and three recorders was able to take the observations on 268 gauge lines and record and work up the data as the test progressed. This involved placing the load in four increments and removing it in two increments and the test itself covered a period of six days. To make an adequate report of such a test is itself quite a task, and the expense of this item is considerable. A much smaller amount of work will give special information on a few matters. The data at hand indicate that a thorough test may cost as much as $1500 for all items and in one test mentioned the total cost was more than $2000.

III. The Wenalden Building Test.

21. *The Building.*—The Wenalden Building, Fig. 27, is a ten-story reinforced concrete structure at 18th and Lumber streets, Chicago. It was built by the Ferro-Concrete Construction Company, Cincinnati, Ohio, in accordance with the plans and specifications of Howard Chapman, architect. It is now occupied by Carson, Pirie, Scott and Company, dry goods merchants, as a warehouse. The building is of the beam and girder type. The floor panels are 15 ft. by 20 ft. The gird-

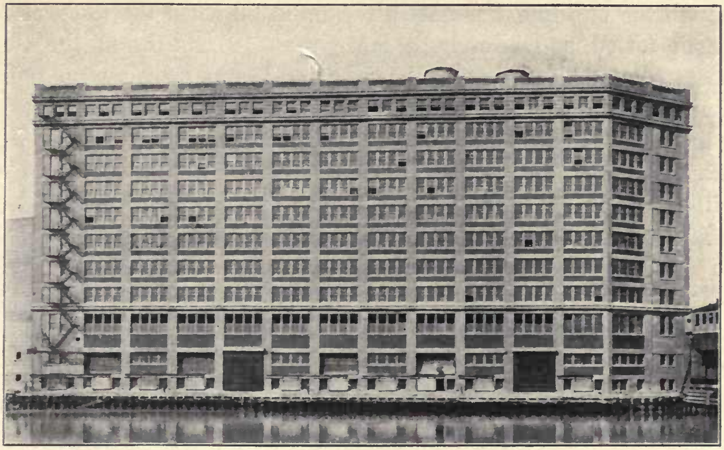

FIG. 27. THE WENALDEN BUILDING.

ers are placed between columns in the short direction. Floor beams extend the long way of the panel, there being two intermediate beams built into and supported by the girders and a column beam built into and supported by the columns. The floor, 3⅞ in. thick (including the top finish), was built continuously with the beams and girders.

The reinforcement is of the form used by the Ferro-Concrete Construction Company. The main reinforcing bars (twisted bars) are carried along the bottom of the beam from the end of a panel to a point beyond the middle of the panel, where they are bent up to the top of the beam and carried horizontally to a corresponding point on the other side of the support, then bent down and continued along the bottom of the beam to the end of the next panel, these reinforcing bars thus having a length of two panels. In the intermediate beams at the bottom and middle there are four rods ¾ in. square and in the side beams one rod ¾ in. square and three rods ⅝ in. square. In the girders there are four rods ⅞ in. square, the disposition of which is similar to that in the beams. By this plan there is twice as much of the main reinforcement in the bottom of the beam or girder at the middle of the span as there is at the top over the supports, except that four ⅜ in. square rods placed in the floor slab are also available for end reinforcement of the intermediate beams. The beams are 6¼ in. and the girders 7½ in. wide. The general position of the reinforcement is shown in Fig. 28. The position of the vertical stirrups is not shown.

The contractors report that the concrete was composed of one part portland cement, 2 parts torpedo sand, and 4 parts crushed limestone.

Although the building was not fully completed when the test was made, the floor tested had been built more than 12 months at the time of the test. The work of cutting the floor and beams for inserting points of measurement proved to be very difficult and showed the concrete to be very hard and of excellent quality.

22. *Method of Testing.*—The test was made on the first floor of the building. This was the only one which could be reached with the loading material. The space chosen was one freest from openings and other

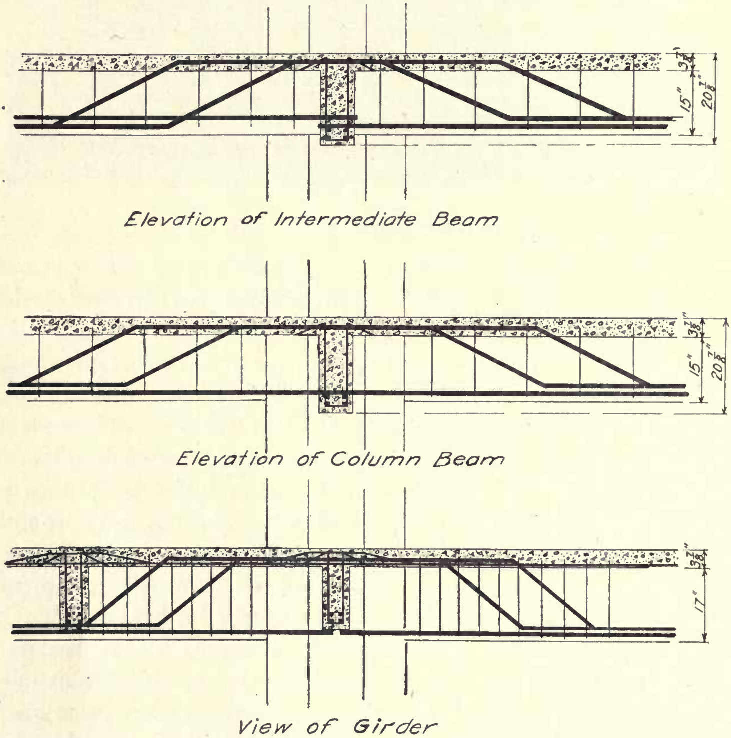

FIG. 28. GENERAL POSITION OF REINFORCEMENT IN WENALDEN BUILDING.

irregularities of construction. At various points at the top and bottom of beams, holes were cut into the concrete until the reinforcement was bared and gauge holes were drilled in the bars 6 in. or 10 in. apart for use in inserting the instruments with which the measurements of elongation were made. Where stresses in the concrete were to be measured, holes were cut in the concrete and short pieces of steel were set in plaster of paris. Gauge holes were drilled in these steel inserts in such a way as to give gauge lines 6 in. or 10 in. long. The position of these points is shown in Fig. 29 and 30. For the work of measuring deflections, steel balls were affixed to the under side of beams and girders at various places and other balls were placed about 7 in. lower on supports which had been

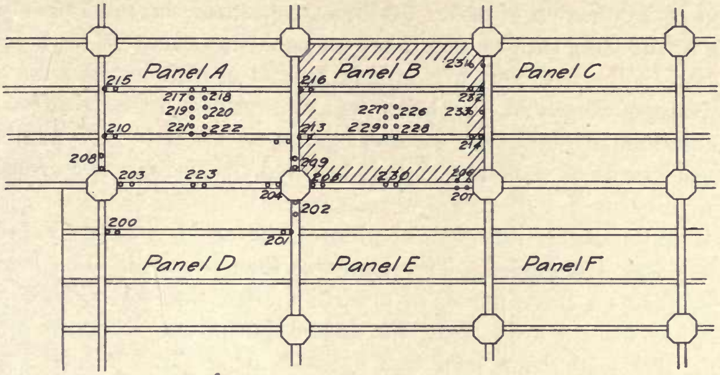

FIG. 29. PLAN SHOWING LOCATION OF GAUGE LINES ON UPPER SIDE OF FLOOR.

built up independently of the observing platforms. A number of these points of deflection were used to determine the inflection points of the beams.

For any observation several instrument readings, usually five, were taken on each gauge line and these were averaged. Measurements were

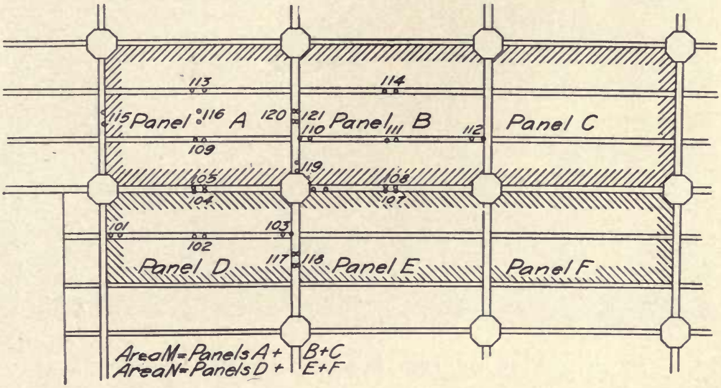

FIG. 30. PLAN SHOWING LOCATION OF GAUGE LINES ON UNDER SIDE OF FLOOR.

made on the standard bar before and after each series of observations to permit corrections for instrumental changes.

23. *Method of Loading.*—The floor was designed for a live load of 200 lb. per sq. ft. and the test load was made 400 lb. per sq. ft. over the panels loaded.

The loading was done by piling brick and bags of cement in piers separated by aisles in such a way as to give access for points of measurements and to prevent arching effects influencing the tests. The load was put on in increments of about 80 lb. per sq. ft. of the total panel area, and a set of observations was taken at each increment of load. Brick was used in the first part of the loading and cement in the later work. The average weight of the brick was determined by weighing a considerable number, and such care was given to determine the number of brick and sacks of cement that it is believed the weights are accurately known.

The following is the general plan of the test. A single panel (*B*, Fig. 29) was first loaded. This load was then removed. The load was then applied on three panels in tandem (*ABC*, Fig. 30). These three panels are termed area *M* in the load-deformation diagrams. Then, leaving this load on, a load was applied along three adjacent panels (*DEF*) covering two-thirds of the width and making in all the equivalent of five loaded panels. This portion of these three panels is termed area

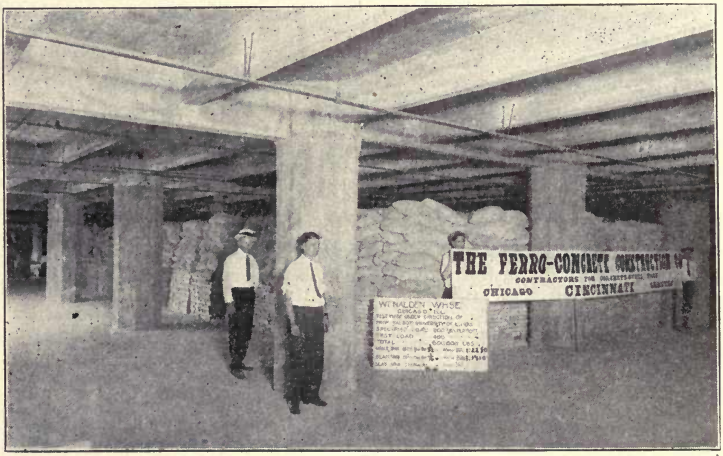

Fig. 31. View of Test Load in Wenalden Building.

N in the load-deformation diagrams. The load was then taken off by increments. The total weight of the load used was 600 000 lb. Fig. 31 is a view of the test load.

A further test was made by loading a single panel at the end of the building, a so-called wall panel.

The loading was begun Monday, July 10, 1911, the loading of five panels was finished at noon on the following Friday, and unloading was completed on Monday, July 17. The schedule of loading is given in Table 4. The wall panel was loaded on Friday and Saturday, July 14 and 15. The unloading of this panel was finished August 1.

24. *The Deformations and Stresses.*—The results of observations for various gauge lines are plotted in Fig. 32, 33 and 34. From a

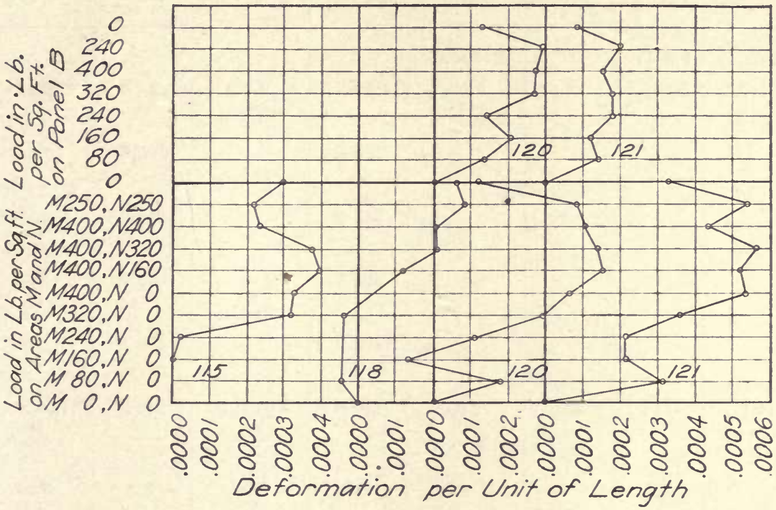

FIG. 32. LOAD-DEFORMATION DIAGRAMS FOR UNDER SIDE OF GIRDER AT MIDDLE.

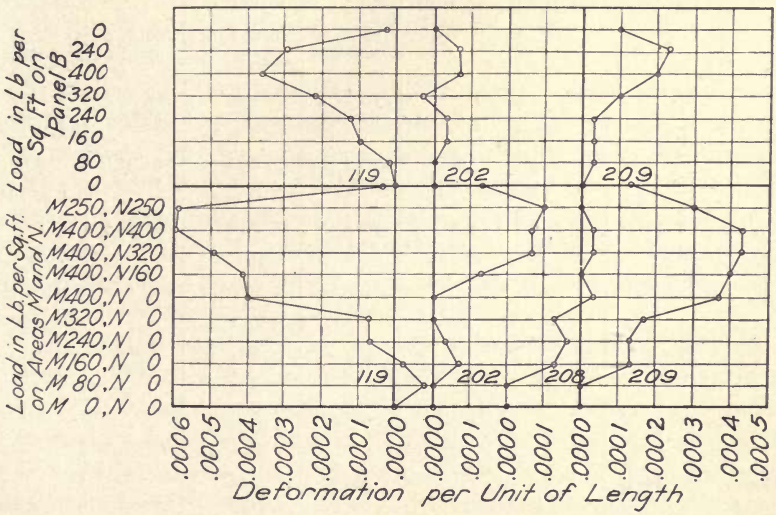

FIG. 33. LOAD-DEFORMATION DIAGRAMS AT END OF GIRDER.

TABLE 4.

Schedule of Loading Operations.

Day	Date	Observations		Loading		Observations	
		Load lb. per sq. ft.	Hours	lb. per sq. ft.	Hours	Load lb. per sq. ft.	Hours
Sunday....	7–9–11	0	5.45 P. M.				
Monday....	7–10–11	0	7.00 to 8.00 A. M.	80 *B*	8.00 to 8.50 A. M.	80 *B*	8.50 to —— A. M.
				160 *B*	—— to 10.25 A. M.	160 *B*	10.25 to —— A. M.
				240 *B*	—— A. M. to 12.10 P. M.	240 *B*	12.10 to —— P. M.
				320 *B*	—— to 3.00 P. M.	320 *B*	3.00 to —— P. M.
				400 *B*	—— to 5.10 P. M.	400 *B*	5.10 to —— P. M.
Tuesday....	7–11–11	400	6.00 to 8.00 A. M.	240 *B*	8.00 A. M. to ——	240 *B*	11.00 A. M. to ——
				0 *B*	12.30 to 3.30 P. M.	0 *B*	3.30 to 4.00 P. M.
Wednesday.	7–12–11	0	6.45 to —— A. M.	80 *A*, *B* and C.	8.00 to 10.00 A. M.	80 *A*, *B* and C.	10.00 to 10.45 A. M.
				160 *A*, *B* and C.	10.45 A. M. to 1.00 P. M.	160 *A*, *B* and C.	1.00 to —— P. M.
				240 *A*, *B* and C.	1.30 to —— P. M.	240 *A*, *B* and C.	—— to 2.45 P. M.
				320 *A*, *B* and C.	3.20 to 5.00 P. M.	320 *A*, *B* and C.	5.00 to 6.30 P. M.
Thursday...	7–13–11	320 *A*, *B* and C.	6.30 to 7.45 A. M.	380 *A*, *B* and C.	8.00 to 9.45 A. M.	380 *A*, *B* and C.	9.45 to 10.30 A. M.
				400 *A*, *B* and C.	10.30 to 11.15 A. M.	400 *A*, *B* and C.	11.15 A. M. to 12.00 M.
				400 *M* 160 *N*	11.30 A. M. to ——	400 *M* 160 *N*	3.15 to 3.45 P. M.
				400 *M*	3.45 to 4.30 P. M.		
Friday.....	7–14–11			320 *N*	8.00 to 9.30 A. M.		

TABLE 4.

SCHEDULE OF LOADING OPERATIONS—*Continued.*

Day	Date	Observations		Loading		Observations	
		Load lb. per sq. ft.	Hours	lb. per sq. ft.	Hours	Load lb. per sq. ft.	Hours
Friday.....	7–14–11			400 *M* 400 *N*	10.30 to 11.30 A. M.	400 *M* 400 *N*	11.30 A. M. to 12 M.
				80 *G*	2.00 to 3.00 P. M.	80 *G*	3.40 to —— P. M.
				160 *G*	—— to 5.00 P. M.	160 *G*	5.00 to 5.30 P. M.
Saturday...	7–15–11			250 *M* 250 *N*	8.00 to —— A. M.	250 *M* 250 *N*	11.40 A. M. to 1.30 P. M.
				240 *G*	8.15 to 9.00 A. M.	240 *G*	9.00 to —— A. M.
				320 *G*	8.40 to —— A. M.	320 *G*	9.50 to 10.00 A. M.
				380 *G*	—— to 10.45 A. M.	380 *G*	10.45 to 11.15 A. M.
				400 *G*	—— to 11.40 A. M.	400 *G*	—— A. M. to 12.00 M.

study of these it is readily seen that there are irregularities in the measurements and that the general trend of some of the lines must be taken rather than absolute values.

In translating from unit-deformation to unit-stress the modulus of elasticity of steel has been taken at 30 000 000 lb. per sq. in. and that of the concrete has been assumed to be 3 000 000 lb. per sq. in. For simplicity the straight-line stress-deformation relation for concrete has been assumed, though it is evident that this relation does not hold for the higher stresses and that calculated stresses based upon this assumption are in excess of the actual stress. The interpreted stress for a number of gauge lines is recorded in Table 5.

Table 6 gives calculated stresses and calculated bending moment coefficients. The first line of each set gives the calculated stresses in the reinforcement and in the concrete based upon the value of the bending moment quite commonly assumed in design calculations, 1/12 Wl, where W is the total distributed load on the beam and l is the span length. These are printed in italics. In these cases the span length was taken as

TABLE 5.

Stress Indications in Wenalden Building Test.

Stresses are given in pounds per square inch.

	Gauge Line.	Single Panel.	Three Panels.	Five Panels.
Reinforcement at end of girder................	208	......	8 000	7 000
	209	7 000	12 000	13 000
	202	......	......	9 000
Reinforcement at middle of girder.............	115	......	10 000	11 000
	120	8 000	14 000	17 000
	121	6 000	16 000	17 000
Concrete at end of girder.....................	119	1 100	1 600	2 200
Reinforcement at end of intermediate beam.....	210	......	9 000	10 000
	211	2 000	9 000	14 000
	212	9 000	14 000	14 000
	213	9 000	16 000	16 000
	214	11 000	13 000	13 000
	216	13 000	13 000	14 000
Reinforcement at middle of intermediate beam..	109	......	14 000	16 000
	111	8 000	7 000	11 000
	113	......	16 000	16 000
	114	6 000	11 000	11 000
Concrete at end of intermediate beam..........	110	......	......	1 300
	112	1 500	1 700	2 000
Concrete at middle of intermediate beam.......	217	Low	Low	Low
	222	"	"	"
	229	"	"	"

TABLE 6.

Maximum Stresses and Moment Coefficients in Wenalden Building Test.

Stresses are given in pounds per square inch.

Member	Reinforcement		Concrete	
	Stress	Coefficient	Stress	Coefficient
Girder, End............................	*44 000*	*1/12*	*1 700*	*1/12*
" End..................................	13 000	0.024	2 200	0.106
" Middle...............................	*19 000*	*1/12*	*420*	*1/12*
" Middle...............................	17 000	0.075	.....	..
Intermediate Beam, End.................	*36 000*	*1/12*	*1 900*	*1/12*
" End..................................	16 000	0.037	2 000	0.088
" Middle...............................	*22 000*	*1/12*	*440*	*1/12*
" Middle...............................	16 000	0.06	.....	..
Column Beam, End.......................	*69 000*	*1/12*	.....	..
" End..................................	15 000	0.018	.....	..
" Middle...............................	*26 000*	*1/12*	.....	..
" Middle...............................	11 000	0.035	.....	..

3 in. longer than the clear span. Measurements had been made upon the position of the bars and the depth of the reinforcement, which were not always exactly according to the plans, and the calculations have been

based upon the dimensions found. In the second line of each group the maximum stress obtained by the measurements is given in the column of stresses, and the bending moment coefficient (the coefficient of Wl) corresponding to these stresses is recorded in the adjacent column. In these calculations the common assumptions of design calculations (including the neglect of the tensile strength of the concrete) are followed except that the width of T-beam is taken as equal to the distance from center to center of beams. In calculating the bending moment coefficient from the measured stress, the position of the neutral axis and the value of the moment arm are assumed to be the same as given by the ordinary assumptions. Although the stress in the reinforcement is measured at the surface of a bar of the outer layer, this stress is considered as being the same as that acting at the center of gravity of the group of bars, for the actual variation in the group is unknown and this method will give a bending moment coefficient larger than that found by considering that the stress in the bars of the other layer is smaller.

In the calculations for compressive stresses, the compression reinforcement was considered to take its proportion of the compressive stress though there may be a question whether the embedment in such designs is sufficient to insure this condition.

It will be seen that in the tests with three and five panels loaded the highest stress observed in the reinforcement in the middle of the intermediate beams was 16000 lb. per sq. in. and the highest stress observed at the ends of the beams was 16000 lb. per sq. in. The stresses observed in other bars having similar positions were lower, and probably the highest stress is not representative of the general stresses. However, it may be best to compare on the basis of the highest stresses. The bending moment given in the table as derived from the measured stresses is .06 Wl at the middle of the beam and .037 Wl at the end of the beam. Measurement of the compression of the concrete in this test was less satisfactory than the measurement of the reinforcement deformations, and considerable variation was found at the different points of observation. Not enough gauge lines gave satisfactory measurements to warrant making quantitative conclusions, but the indications of the action of the concrete may be useful. The value of the resisting moment which corresponds to the concrete stresses, on the assumptions made, is .088 Wl for the end of the beam. For the middle of the beam the stresses were small and the indications so irregular that no value of resisting moment can be given.

In the beams at the sides of the panel (column beams) the stresses were in general somewhat lower, but with a full loading a stress of 15000 lb. per sq. in. at the middle of the beam was observed and 11000 lb. per sq. in. at the end.

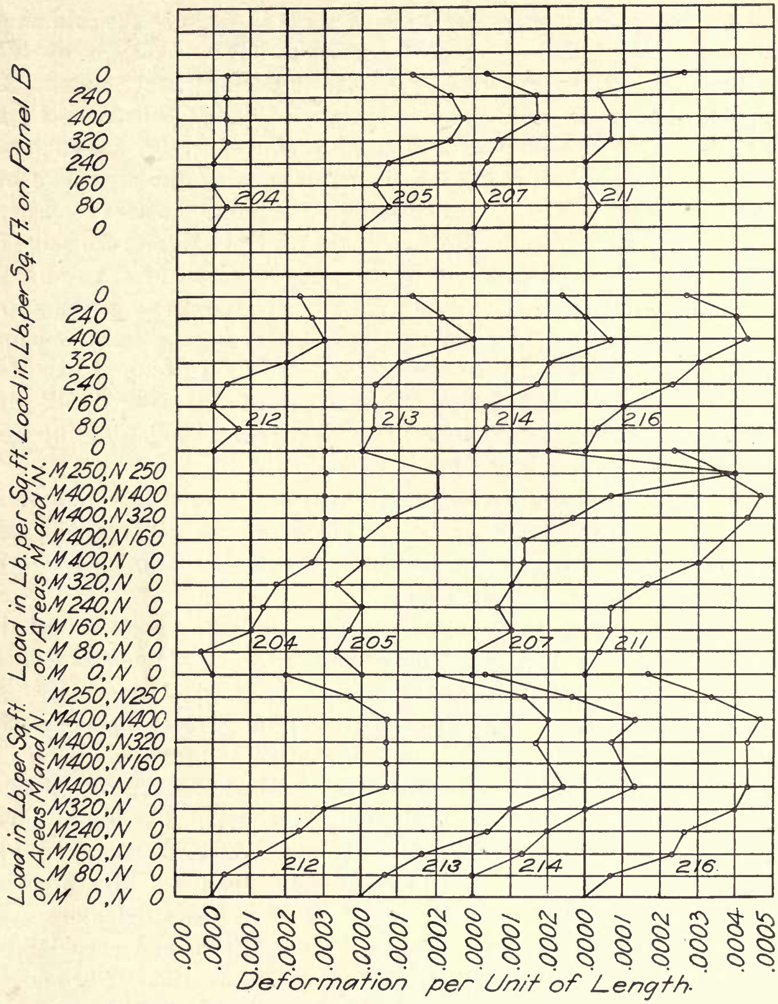

Fig. 34. Load-deformation Diagrams for Upper Side of Beams at End.

Fewer measurements were made on the reinforcement of the girder. A stress of 17 000 lb. per sq. in. was observed at the middle of the girder and 13 000 lb. per sq. in. at the end. On the same assumptions these values correspond to bending moments of .075 Wl and .024 Wl, respectively. The stress in the concrete at the end of the girder was also very high, but the corresponding bending moment (.106 Wl) is not far from the calculated moment for a restrained beam with concentrated load. It should be noted in this connection that the reinforcement is bent down rapidly into the beam from the face of the column, see Fig. 28.

Calculating with the usual assumptions of beam formulas, the total compressive stress in the concrete at the end of the beam is greater than the total tensile stress in the reinforcement. Two elements probably enter into the results, the tensile strength of concrete, which may be considerable as distributed over the width of the floor, and an arching action of the structure. However, it should be noted that the value of the bending moment coefficient derived from the reinforcement stresses at the middle of the beams and girder is not much less than values commonly used and also that the calculated resisting moment developed at the end of the beam based on the concrete stresses is not far from the amount usually assumed.

Attention should be called to the fact that the compressive stress in the concrete, both that calculated from an assumed bending moment coefficient and that calculated from the measured deformation, is much higher than that to be found by the use of the parabolic stress deformation relation and the actual stress will be less than that given in the table.

Measurements were made on the concrete at the top of the floor slabs in a direction parallel with the beams to find the distribution of compressive stresses between beams. These measurements were not fully satisfactory, but within the limits of accuracy of the measurements, no difference in the amount of shortening over the beam and at points between beams could be determined, and the whole floor evidently acted as a part of the compression member of the T-beam so formed.

25. *Test Cracks.*—The surface of the beams and girders had received a white coat, which permitted very fine cracks to be detected, much finer than may be observed on uncoated concrete. In the test, as the load was applied, fine tension cracks in the concrete through the middle of the length of the beam were observable at stresses in the reinforcement corresponding to the stresses at which load cracks are detected in the tests of beams in laboratory work. To an experienced observer development of the cracks was confirmation of the measurements of the low stresses developed in the reinforcement. Upon removal of load most of these cracks closed until they were not visible to the eye.

As the calculated reaction on the end of a girder was upward of 40000 lb., it will be seen that the vertical shearing stresses were very high. Diagonal tension cracks developed in these girders just outside the junction with the intermediate beams, making an angle of nearly 45° with the horizontal. These cracks did not entirely close on the removal of the load. No measurements were taken to determine the diagonal deformations. It seems probable that the restraint at the end of the girder and the tensile strength of the concrete acted to prevent the fuller development of these cracks.

No diagonal cracks were observed in the beams.

26. *Deflections.*—Fig. 35 gives the location of the points at which the deflections were measured. Fig. 36 shows the deflections with a load

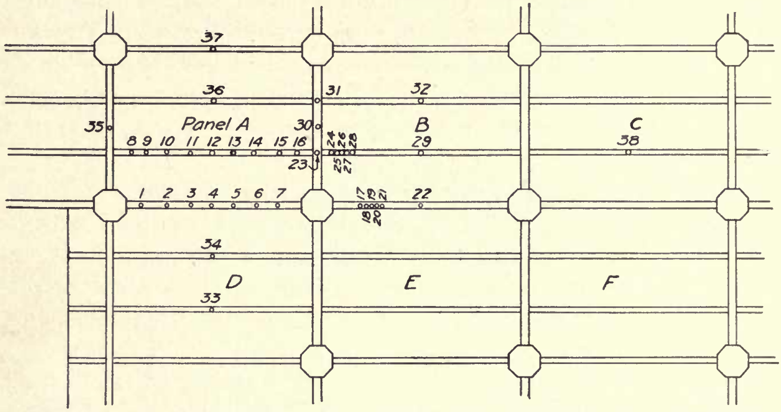

FIG. 35. LOCATION OF DEFLECTION POINTS IN WENALDEN BUILDING.

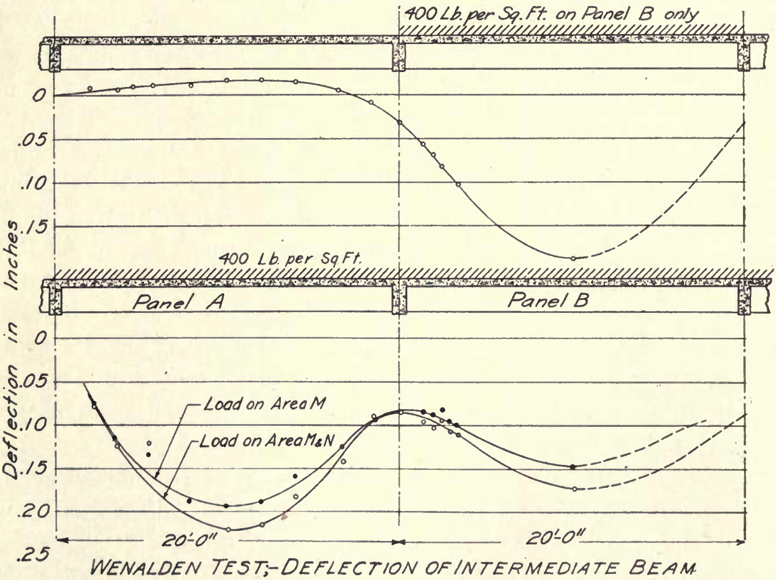

FIG. 36. DIAGRAMS SHOWING DEFLECTION OF INTERMEDIATE BEAM.

of 400 lb. per sq. ft. for points along an intermediate beam, (1) with one panel loaded (panel *B,* Fig. 29) and (2) with three panels loaded (area *M,* Fig. 30), and (3) with five panels loaded (areas *M* and *N,* Fig. 30). As may be expected, the deflection in the middle panel is greater for one panel loaded than when three panels are loaded.

27. *Wall Panel.*—A single wall panel was loaded and observations were taken on the gauge lines which are shown in Fig. 37. No meas-

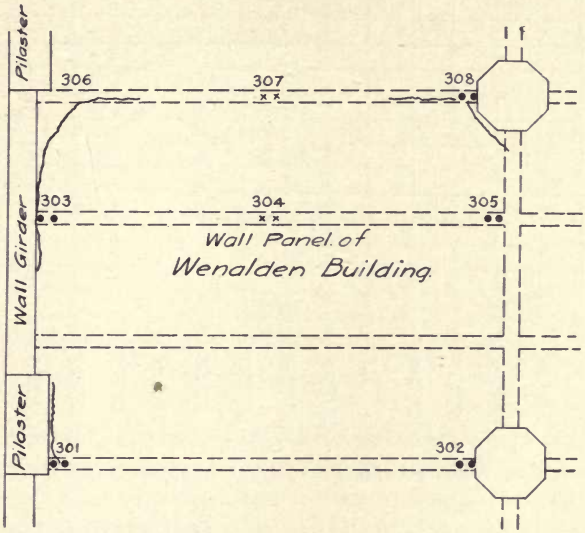

FIG. 37. WALL PANEL TEST; PLAN SHOWING LOCATION OF GAUGE LINES.

urements of the compression in the concrete were made. On the reinforcement only a few gauge lines were used. The observed values are plotted in Fig. 38 and Fig. 39. Because of the small number of gauge lines and because of some of the conditions of the test which were not entirely favorable, the results may not be trustworthy quantitatively as compared with the other tests, but the indications are of interest. There is considerable restraint shown at the ends of the beams, that at the pilaster being about the same as that at the column end and that at the wall being greater than that at the girder end. All of these are nearly as large as the values found in the interior panels. The stress in the middle of the beams is considerably greater than that found in the middle of the beams in an interior panel. The deflections are also greater than for interior panels and the deflection curves are of a different character. It is recognized that there are some apparent inconsisten-

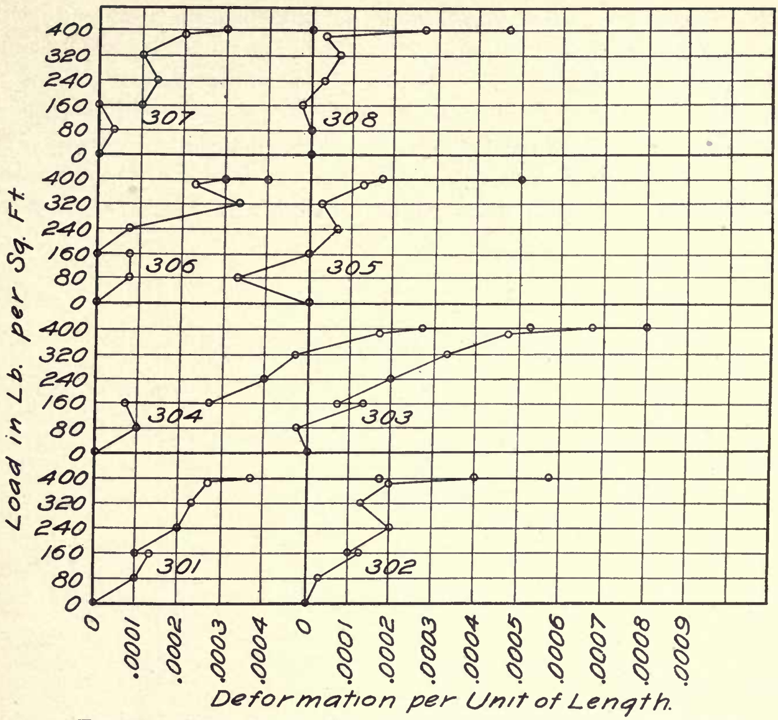

FIG. 38. WALL PANEL TEST; LOAD-DEFORMATION DIAGRAM.

cies in these statements, and the action of wall panels is a matter which should receive full investigation in the future.

28. *Examination of Floor after Test.*—An examination of the floor was made May 6, 1912, to ascertain whether the cutting of the concrete for purposes of observation had caused any permanent disfigurement. On the under surface of slabs and beams where the chances would be greatest for material used for filling the test holes to fall out of place, the concrete was intact. It seems probable that if the surface had been painted over after the repairs were made the patched portions could not have been detected without a minute examination. Although the basement was well lighted in the vicinity of the main test the only indications of the location of cracks were pencil marks where the cracks had been traced to secure ease in sketching their position. These pencil marks had been painted over but showed through the thin coat of white. It is probable that a more minute examination would have detected cracks, but the fact that after removal of the test load not even the diagonal tension cracks were plainly visible bears out the conclusions that the

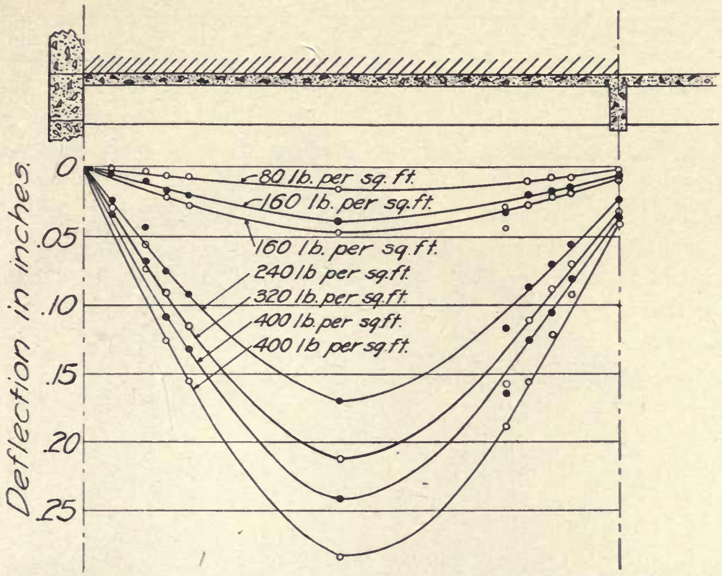

FIG. 39. WALL PANEL TEST; DIAGRAM SHOWING DEFLECTION OF INTERMEDIATE BEAM.

steel stresses caused by the test load were light. The basement under that part of the floor where the wall panel test was made was not so well lighted, hence the examination here was not so significant. On the upper surface of the floor tested, there were cracks which were distinct, but not more so than many which were observed before the test had been made. The area on which the wall panel test was made was inaccessible, being completely covered with merchandise.

IV. THE TURNER-CARTER BUILDING TEST.

29. *The Building.*—The Turner-Carter building (see Fig. 40) is an eight-story reinforced concrete building 60 x 200 ft., located at Willoughby Avenue and Walworth Street, Brooklyn, New York. It was constructed by the Turner Construction Company for the Turner-Carter Company (manufacturers of shoes) in accordance with the plans and specifications of Frank Helmle, architect.

The building is of the beam and girder type. The panels are 17 ft. 4 in. by 19 ft. 6 in. The floor was built continuously with the beams and girders. The girders are 10 in. wide and 24 in. deep including the finished floor. Each panel has two intermediate beams 7 in. wide with a total depth of 18 in. The column beams are the same size as the inter-

FIG. 40. THE TURNER-CARTER BUILDING.

mediate beams. The columns below the test floor are octagonal and are 30 in. from face to face. The position of reinforcement of the beams and girders is shown in Fig. 41. The beams and girders were designed as simple beams, but reinforcement is supplied for continuity, and the construction is such as to give continuity in the beams and girders. The structure was designed for a live load of 150 lb. per sq. ft.

The aggregates were an excellent grade of sand and gravel obtained from the sand banks in Hempstead Harbor on the north shore of Long Island. The gravel ranged in size from 3/8 to 7/8 in. For the beam and girder reinforcement bars having an elastic limit of about 50000 lb. per sq. in. were used. The beams have one 1-in. square bar and two 7/8-in. square bars at the middle and one 1-in. square bar over the support carried about 15 in. beyond the center line of the girder. Ten 3/8-in. round bars placed in the slab are also available for tension reinforcement in the end of the intermediate beams, as is also one T-bar used for supporting the slab reinforcement during construction. The girders have two 1-in. square and three 7/8-in. square bars at the middle, placed in two layers, and two 1-in. square bars over the support carried nearly to the farther face of the column.

The floor tested was constructed July 25 so that at the time of the test, September 10 to 20, 1911, the work was about fifty days old.

30. *Method of Testing.*—The feature of the test, as of the Wenalden test, was the measurement of the deformations in the reinforce-

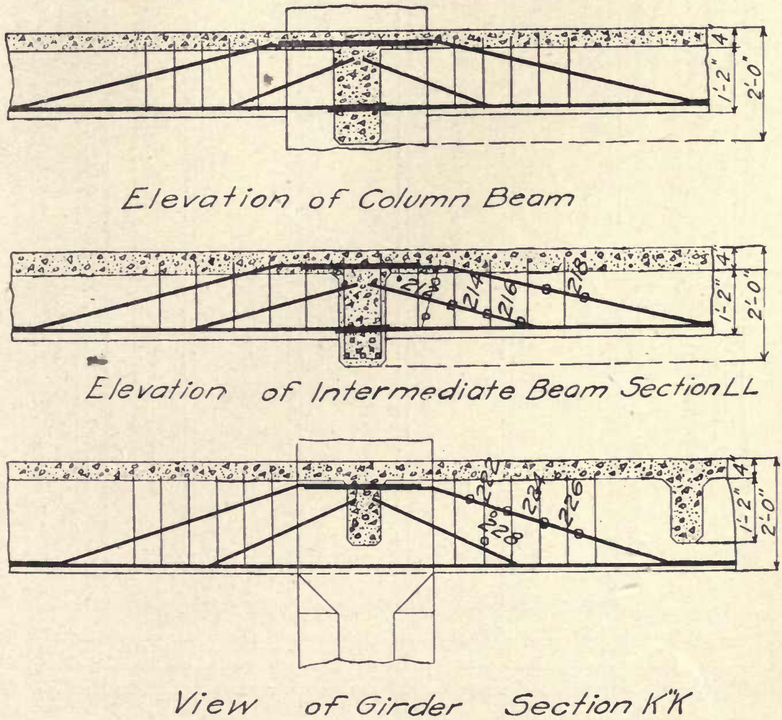

FIG. 41. SKETCH SHOWING REINFORCEMENT OF BEAMS AND GIRDERS AT SUPPORTS.

ment and in the concrete at various points in the girders, beams and slabs. The most important determinations undertaken in the test were the measurement of the compressive deformations in the concrete at and

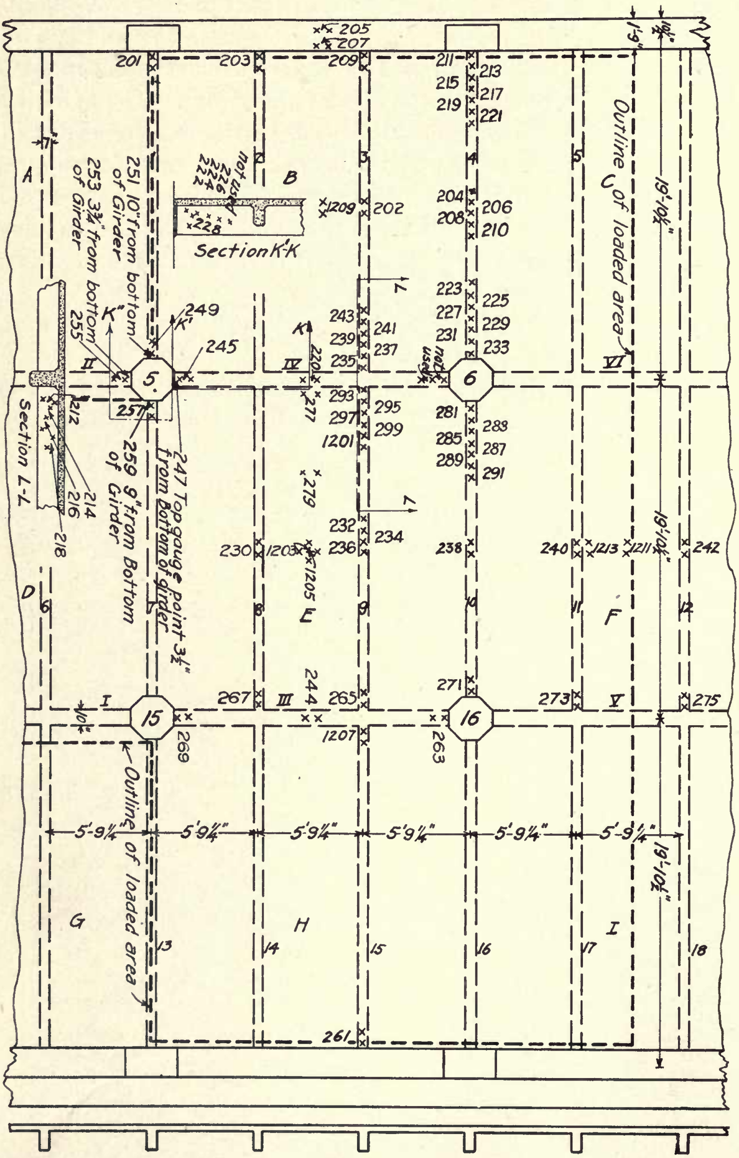

FIG. 42. PLAN SHOWING LOCATION OF GAUGE LINES ON UNDER SIDE OF FLOOR.

near the supports of the beams, the compressive deformations of the concrete at the middle of the beam, and the distribution of these compressive stresses across the top of the slab between beams to determine the extent of T-beam action. The deformation in the reinforcement was

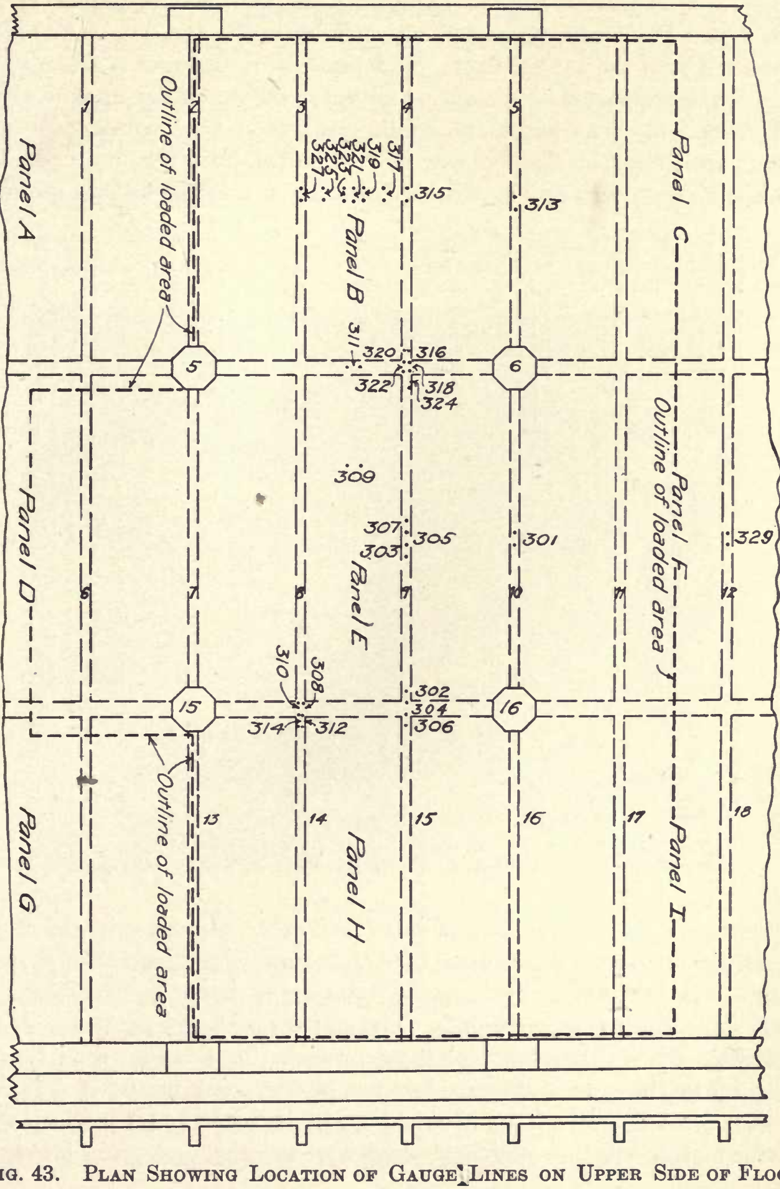

FIG. 43. PLAN SHOWING LOCATION OF GAUGE LINES ON UPPER SIDE OF FLOOR.

measured at the centers of the spans and at the ends and also on the inclined portions of the bent-up bars. Various other measurements which it was thought would throw light upon the action of the structure were taken.

31. *Preparation for the Test.*—A week was used in preparing for the test. Platforms supported by scaffolding for the use of observers were built on the second floor. Independent of this was a framework, which was supported by the second floor, for use in making measurements of deflection. The boxes for holding the sand were constructed, this being facilitated by a power saw located on the second floor. Considerable time was consumed in drilling holes in the concrete to bare the rein-

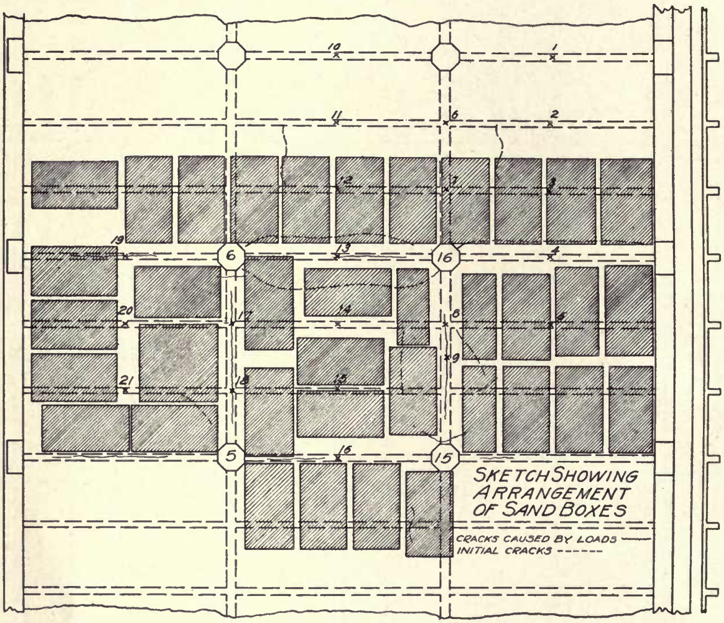

FIG. 44. LOCATION OF SAND BOXES AND FLOOR CRACKS.

forcement. In some cases this was found to be at a considerable depth from the surface. In all nearly two hundred holes were cut in the concrete. Holes were drilled in the reinforcing bars, as heretofore described, for use as gauge points. The gauge length was made 8 in. The position of the gauge lines for the reinforcing bars is shown on Fig. 42 and 43 by the even numbers. For use in the measurement of deformations of the concrete, holes about ½ in. in diameter and 1 in. deep were drilled in the concrete and steel plugs were inserted and set in plaster of paris. Gauge holes for receiving the points of the extensometers were drilled in these plugs with a No. 54 drill. The position of the gauge lines is shown in Fig. 42 and 43 by the odd numbers. The gauge length was 8 in.

The deflections were measured between a steel ball set in the under surface of the beam and a ball attached to the framework previously described. The measurements were made as described in Art. 13.

32. *Method of Loading.*—The test area was on the third floor. The loading material was damp sand which was placed in bottomless boxes. These boxes were of various sizes and were placed in such a way as to give a well distributed load. The general size of the box was 4 ft. 6 in. wide, 8 ft. long and 4 ft. 6 in. deep. Fig. 44 shows the position of the boxes and the test area. Fig. 45 is a view with the sand boxes ready for

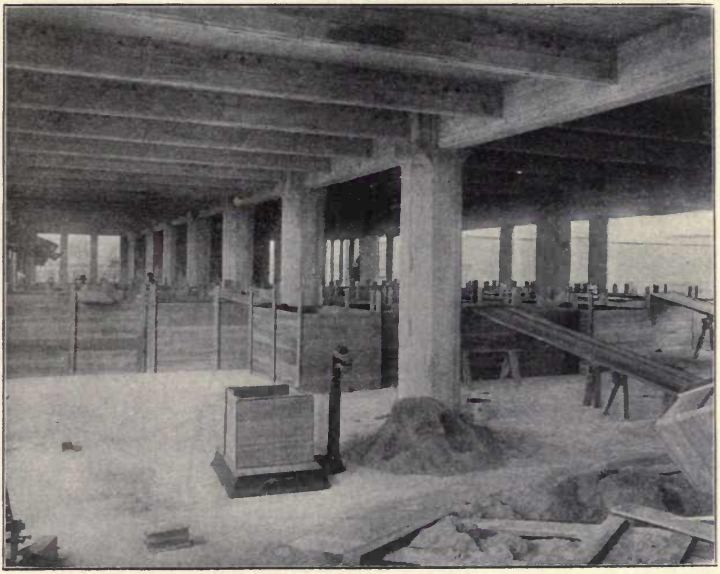

FIG. 45. VIEW OF SAND BOXES.

loading. The boxes were made small enough to permit a good distribution of load even though part of the weight of the sand might be carried by arching and friction down the sides. The test area covered three full panels and parts of four others, in all equivalent to five panels. A loading space was chosen which it was thought would give the fullest stresses over the girders and beams on which the principal measurements were made. In removing the load the outer panels were unloaded first in an attempt to determine the relation between single panel loading and group loading. The load applied was the equivalent of 300 lb. per sq. ft., double the design live load.

Before beginning the test, a calibration of the heaviness of the sand was made by weighing the sand which had been shoveled into a box of 16 cu. ft. capacity placed on the scales. It was found that there was a difference of about 10 per cent in the weight of sand which had been

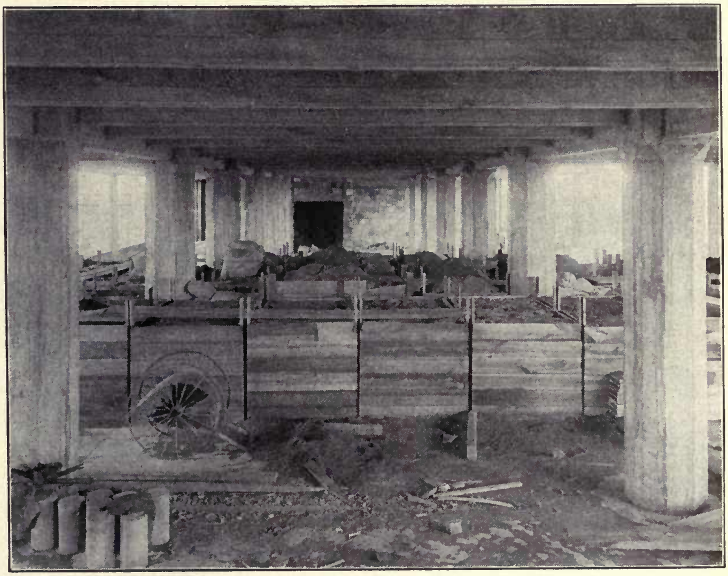

FIG. 46. VIEW OF TEST LOAD IN TURNER-CARTER BUILDING.

thrown in loosely and sand which was packed somewhat. During unloading, the entire contents of three of the sand boxes (about 500 cu. ft.) were weighed. This gave an average of 88.6 lb. per cu. ft., agreeing closely with the weights of the unpacked sand previously weighed, and this value was used in the calculation of loads.

On a part of the area where the boxes were not carried to a sufficient height and where the space was not covered adequately by them, cement in sacks was used as loading material.

The supply of sand for the loading had previously been delivered on the same floor, the piles being kept at least one panel away from the location of the test area, and this was distributed over sufficient floor space that the stresses in the beams of the test area could not be affected. In applying the load the sand was wheeled in barrows and dumped into the boxes. As the sand was placed, the sides of the boxes were rapped to break the adhesion of the sand. Some leveling of the sand in the boxes was done, but there was little compacting by tramping or otherwise.

33. *Making the Test.*—A very important element of a test of this kind is the initial observation for fixing the zero point of the test readings. Three sets of observations for a number of gauge lines were made

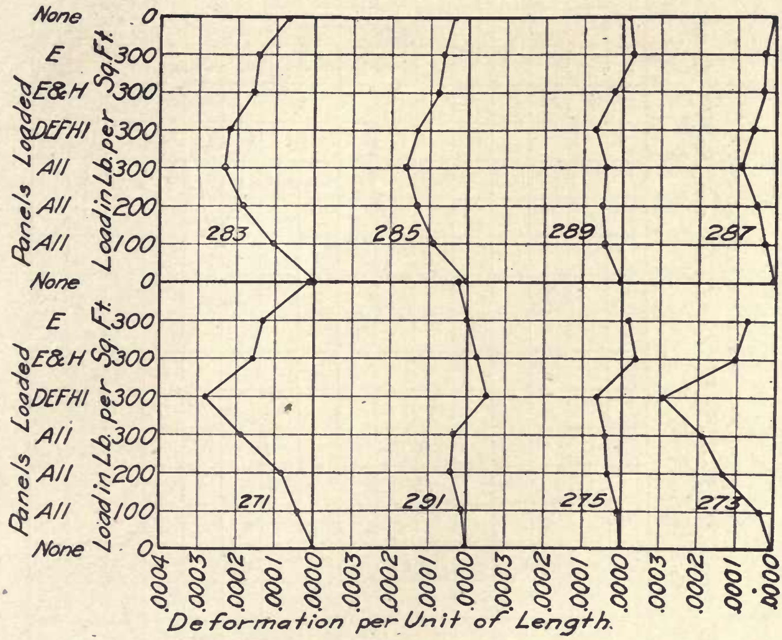

FIG. 47. LOAD-DEFORMATION DIAGRAMS FOR UNDER SIDE OF BEAMS AT END.

before the beginning of the test, on the afternoon of September 10 and the forenoon of September 11. Where discrepancies were found new observations were made. Even with this number of observations there are uncertainties in some initial readings. Experience confirms the view that before any load is placed the initial readings which have been taken should be worked up and observations repeated until all discrepancies and uncertainties have been removed.

Readings were taken immediately after the completion of each increment of load and again immediately before the beginning of placing another increment of load. This usually corresponded with evening readings and morning readings. A series of readings was also taken with the full test load on. These extended over a period of 48 hours. A similar method was used in the process of removing the load.

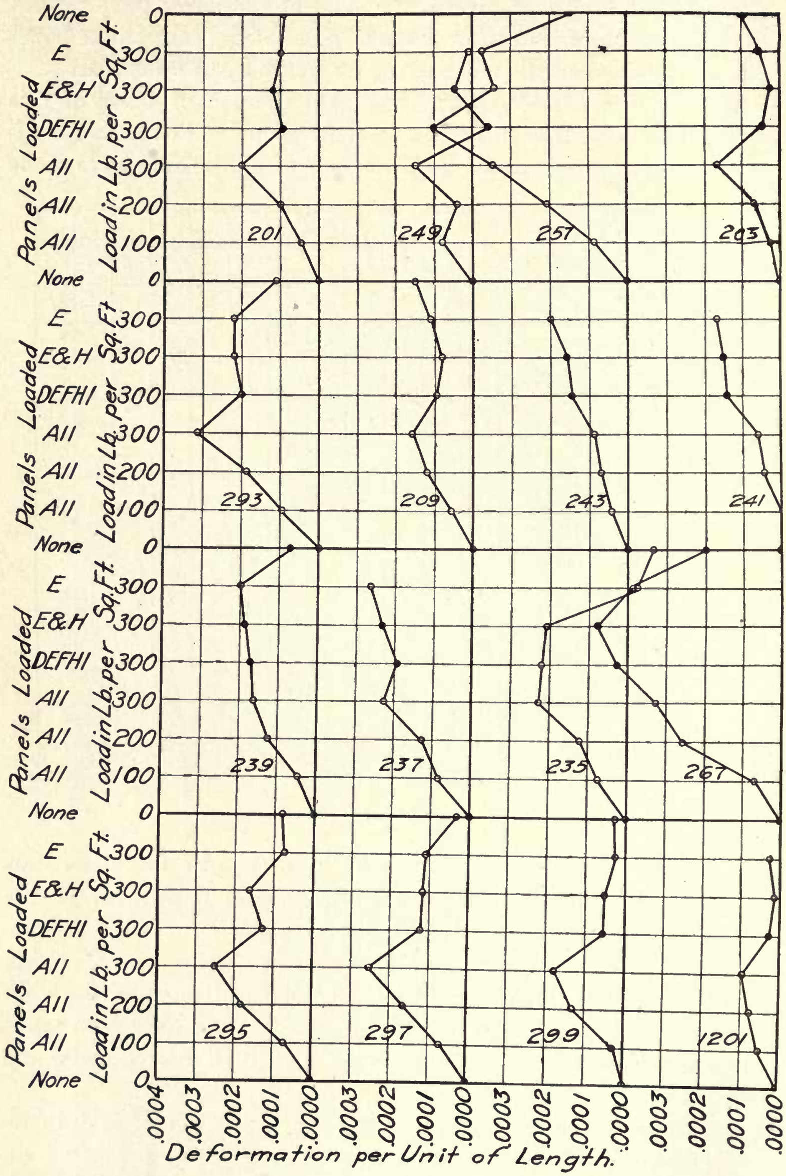

FIG. 48. LOAD-DEFORMATION DIAGRAMS FOR UNDER SIDE OF BEAMS AT END.

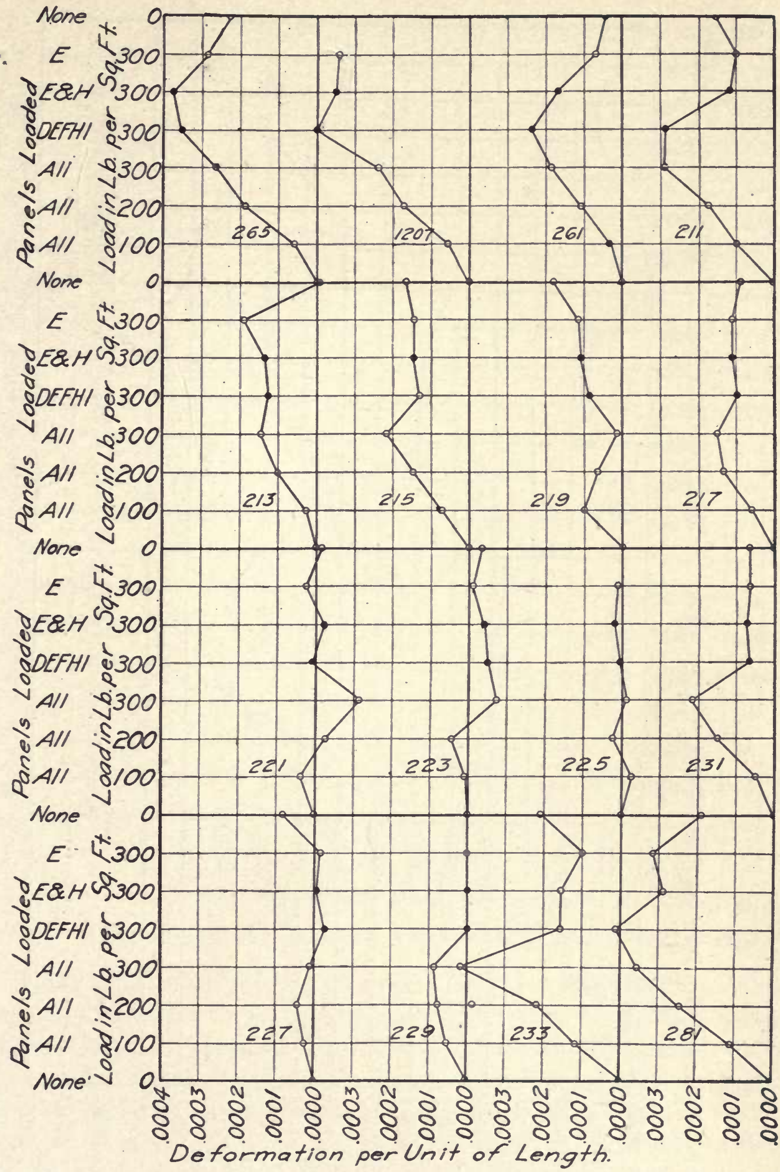

FIG. 49. LOAD-DEFORMATION DIAGRAMS FOR UNDER SIDE OF BEAMS AT END.

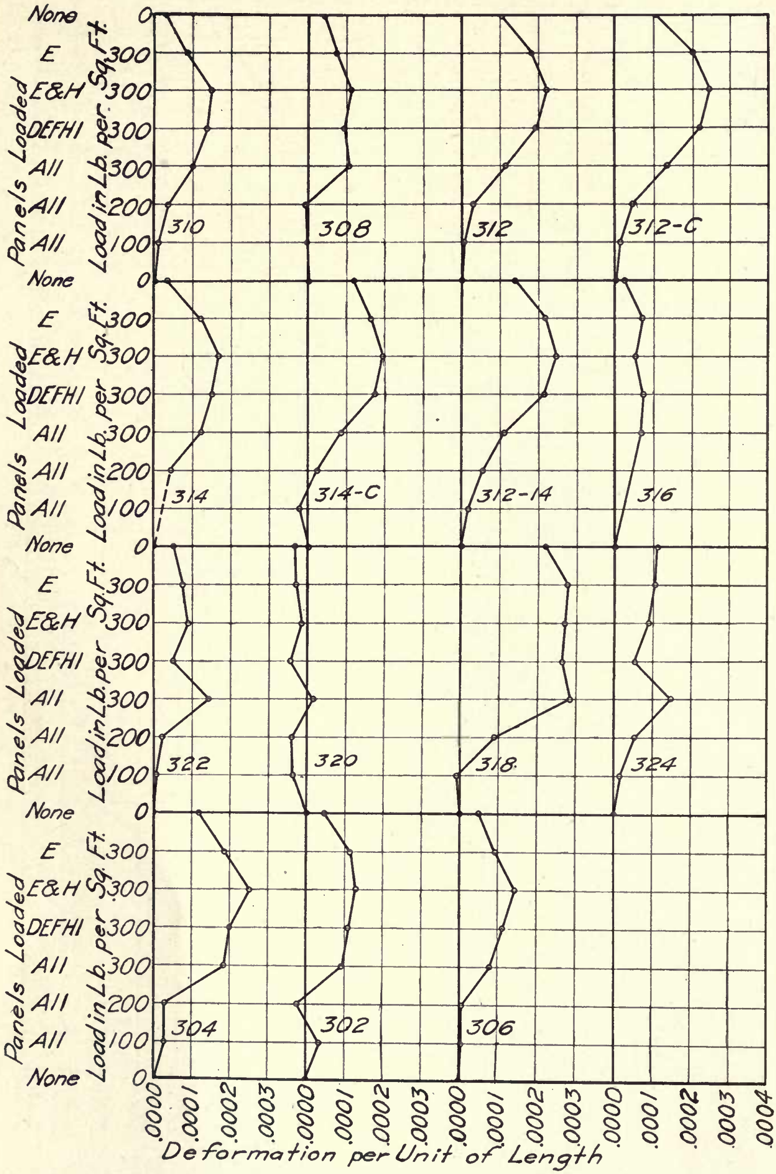

FIG. 50. LOAD-DEFORMATION DIAGRAMS FOR UPPER SIDE OF BEAMS AT END.

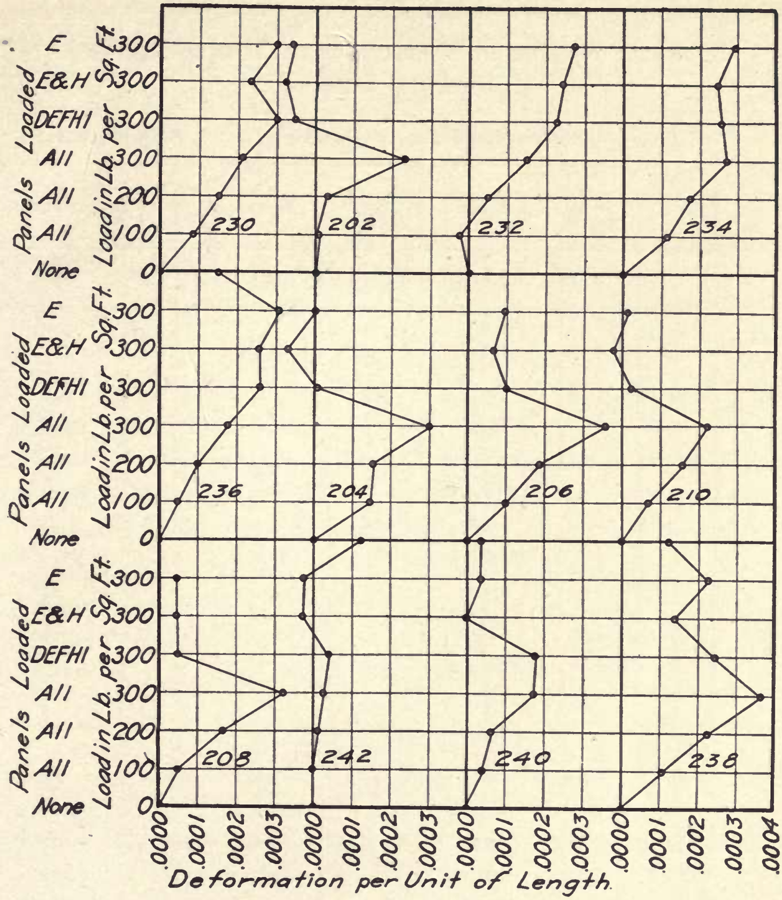

FIG. 51. LOAD-DEFORMATION DIAGRAMS FOR UNDER SIDE OF BEAMS AT MIDDLE.

Table 7 shows the loading schedule. The load was applied in increments of 100 lb. per sq. ft. based upon the whole test area. The application of the load consumed three days. The full load was left on 48 hours. The unloading schedule is shown also in Table 7. In the unloading, the load on panels *B* and *C* was first removed, then the load on panel *D, F* and *I,* followed by the removal of the load on panel *H.* Fig. 46 is a view at a load of 300 lb. per sq. ft. over the test area. The total load was over 500 000 lb.

34. *Deformations and Stresses.*—The results of observations on various gauge lines for the beams and girders are plotted in Fig. 47 to 54. Fig. 55 gives the deformations in the concrete on the under side of the

TABLE 7.

SCHEDULE OF LOADING OPERATIONS IN TURNER-CARTER BUILDING TEST.

Day	Date	Observations		Loading		Observations	
		Load lb. per sq. ft.	Hours	lb. per sq. ft.	Hours	Load lb. per sq. ft.	Hours
LOADING SCHEDULE.							
Sunday....	9-10-11	0	12 M. to 2 P. M.				
Monday....	9-11-11	0	7.20 A. M. to 12 M.	100	1.30 to 6.00 P. M.	100	6.10 to 8.00 P. M.
Tuesday....	9-12-11	100	6.30 A. M. to 8.15 A. M.	200	10.30 A. M. to 3.00 P. M.	200	3.10 to 5.30 P. M.
Wednesday.	9-13-11	200	6.20 to 8.20 A. M.	300	9.00 A. M. to 3.30 P. M.	300	3.50 to 5.50 P. M
						300	10.30 to 11.30 P. M.
Thursday...	9-14-11	300	8.00 to 8.30 A. M.			300	3.00 to 3.30 P. M.
UNLOADING SCHEDULE.							
Friday.....	9-15-11	300	7.30 to 9.30 A. M.	300 on *D*, *E*, *F*, *H* and *I*.	3.30 to 7.30 P. M.	300 on *D*, *E*, *F*, *H* and *I*.	8.00 to 8.30 P. M.
Saturday...	9-16-11	300 on *D*, *E*, *F*, *H* and *I*.	7.20 to 9.15 A. M.	300 on *E* and *H*.	9.30 to 11.45 A. M.	300 on *E* and *H*.	6.30 to 8.00 P. M.
Monday....	9-18-11	300 on *E* and *H*.	6.15 to 9.20 A. M.	300 on *E* only.	9.30 A. M. to 12.00 M.	300 on *E* only.	12.15 to 1.50 P. M.
							4.15 to 8.00 P. M.
Tuesday....	9-19-11					300 on *E* only.	4.50 to 6.50 P M.
Wednesday.	9-20-11	300 on *E* only.	8.30 A. M. to 12.30 P. M.	Zero.	1.00 to 3.40 P. M.	Zero on all panels.	4.00 to 5.40 P. M.

floor slab and Fig. 56 those on the upper side. Fig. 57 records measurements made on the bent-up bars and stirrups.

As already stated, the location of the gauge lines is shown on Fig. 42 and 43, the odd numbers referring to measurement on the concrete, the even numbers to measurement on the reinforcement. The numbers in the two hundreds are gauge lines on the under side or second story side, and the numbers in the three hundreds are on the upper side or third story side.

Stresses and bending moment coefficients are tabulated in Tables 8

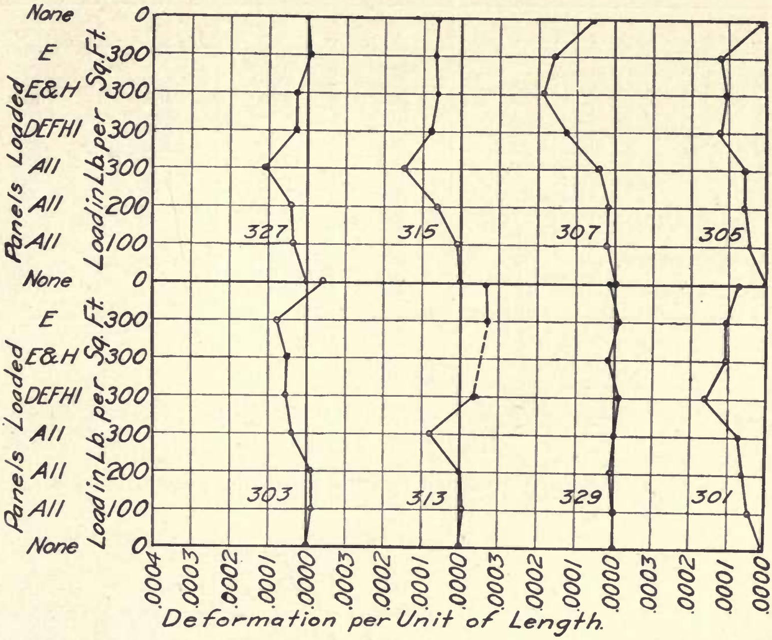

FIG. 52. LOAD-DEFORMATION DIAGRAMS FOR UPPER SIDE OF BEAMS AT MIDDLE.

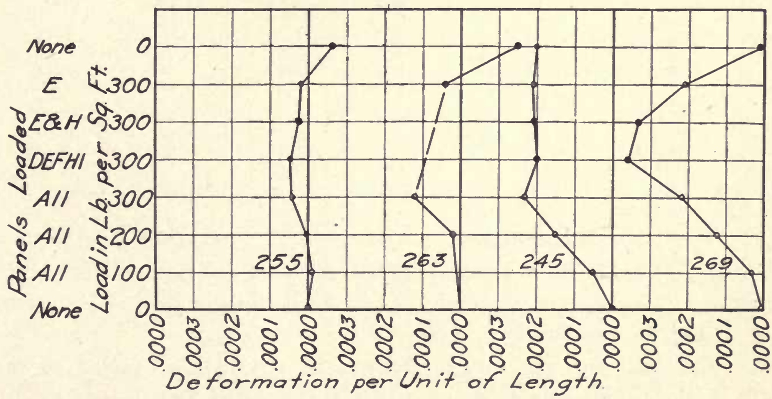

FIG. 53. LOAD-DEFORMATION DIAGRAMS FOR UNDER SIDE OF GIRDERS AT END.

TABLE 8.

Stress Indications in Turner-Carter Building Test.

Stresses are given in pounds per square inch.

Member	Gauge Line	Reinforcement	Gauge Line	Concrete
End of girder	...	......	269	900
Middle of girder	220	8 000	311	Little
"	244	9 000	...	......
End of beam	304	8 000	265	1 100
"	318	8 000	267	1 100
"	310	4 000	281	1 000
"	...	......	293	800
Middle of beam	202	7 000	301	350
"	206	11 000	305	350
"	230	9 000	313	200
"	234	8 000	315	300
"	236	8 000	...	......
"	238	11 000	...	......
"	240	5 000	...	......
Bent up bar in girder	222	5 000	...	......
"	224	5 000	...	......
Bent up bar in beam	214	—3 000	...	......

TABLE 9.

Maximum Stresses and Moment Coefficients in Turner-Carter Building Test.

Stresses are given in pounds per square inch.

Member	Reinforcement		Concrete	
	Stress	Coefficient	Stress	Coefficient
Girder, End	*31 000*	*1/12*	*1 200*	*1/12*
" End	......	..	900	0.06
" Middle	*12 500*	*1/12*	*300*	*1/12*
" Middle	8 000	0.05	Little	.....
Intermediate Beam, End	*21 500*	*1/12*	*1 300*	*1/12*
" End	8 000	0.03	1 100	0.07
" Middle	*18 500*	*1/12*	*380*	*1/12*
" Middle	11 000	0.05	350	0.077
Column Beam, End	*19 600*	*1/12*	*1 200*	*1/12*
" End	......	..	950	0.064
" Middle	*17 000*	*1/12*	*350*	*1/12*
" Middle	10 000	0.05	225	0.054

and 9. The stresses calculated on the basis of a bending moment coefficient of 1/12, the more usual one in designing are printed in italics.

The suggestions given for caution and care in interpreting measurements should be applied to this test.

35. *Beams.*—For the tensile stresses in the reinforcement at the middle of the intermediate beams at the full load of 300 lb. per sq. ft., the highest stress observed was 11000 lb. per sq. in. and the average

stress recorded may be said to be 8500 lb. per sq. in. At the ends of the intermediate beams, the highest stress observed in the reinforcement was

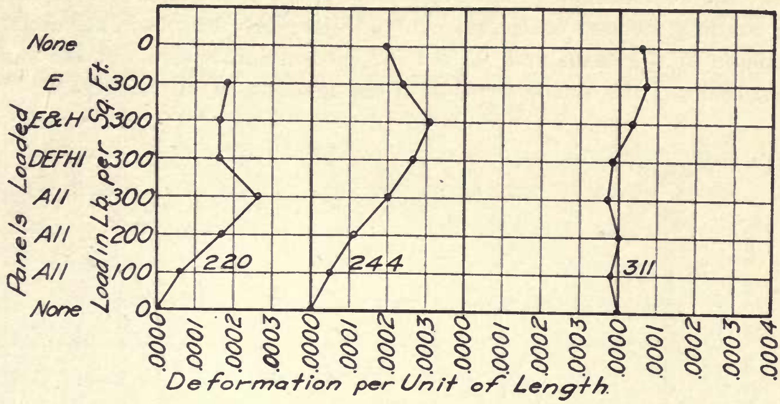

FIG. 54. LOAD-DEFORMATION DIAGRAMS FOR UPPER SIDE AND UNDER SIDE OF GIRDERS AT MIDDLE.

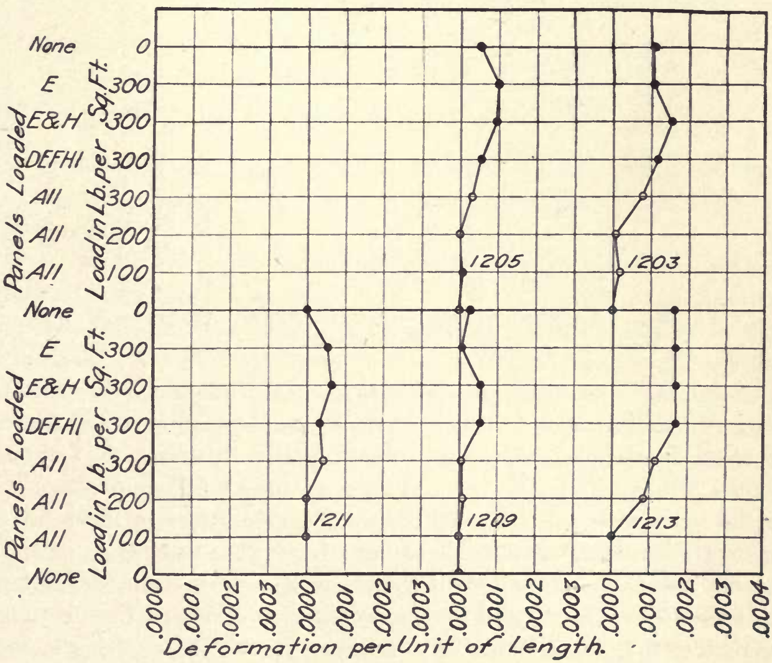

FIG. 55. LOAD-DEFORMATION DIAGRAMS FOR CONCRETE ON UNDER SIDE OF SLAB.

8000 lb. per sq. in., and the general value may be said to be 7500 lb. per sq. in. Using the assumptions for resisting moment ordinarily taken in design calculations, these stresses may be considered to correspond to a bending moment coefficient of .05 *Wl* for the maximum stress at the middle of the beam and .03 *Wl* for the maximum stress at the end of the beam, if the tensile strength of the concrete be not considered.

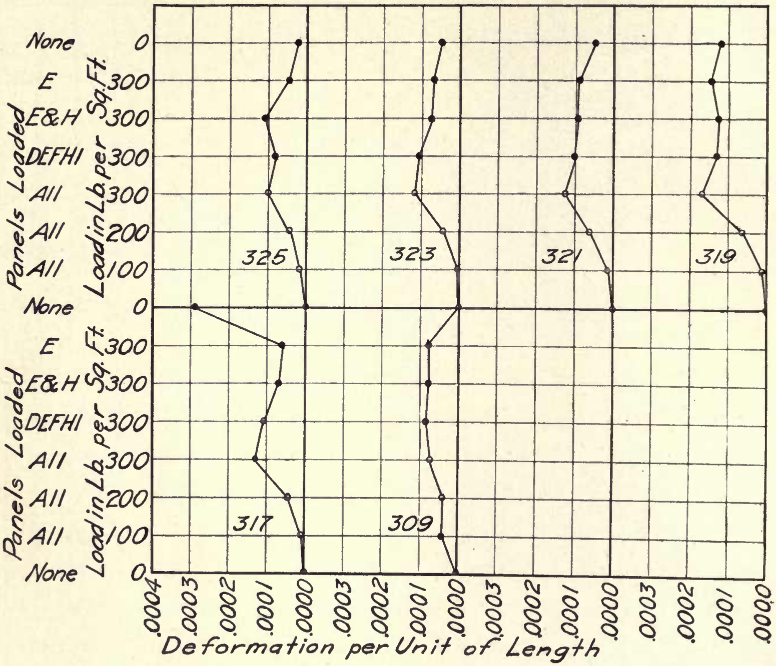

FIG. 56. LOAD-DEFORMATION DIAGRAMS FOR CONCRETE ON UPPER SIDE OF SLAB.

Assuming a modulus of elasticity for the concrete of 2 500 000 lb. per sq. in., the concrete on the compression side of the beams at the middle showed a compressive stress of 350 lb. per sq. in. and at the end of the beam 1100 lb. per sq. in. It is apparent that the total compressive stress in the concrete is greater than the total tensile stress in the reinforcement of the beams. A possible explanation is that end thrust exists, involving so-called arch action in the beams and floor structure, and that the tensile stress is relieved by the presence of this thrust. The tensile strength of the concrete must have a large effect on the resisting moment. The coefficient for *Wl* in the expression for bending moment, necessary to give a compressive stress equal to the maximum measured in the con-

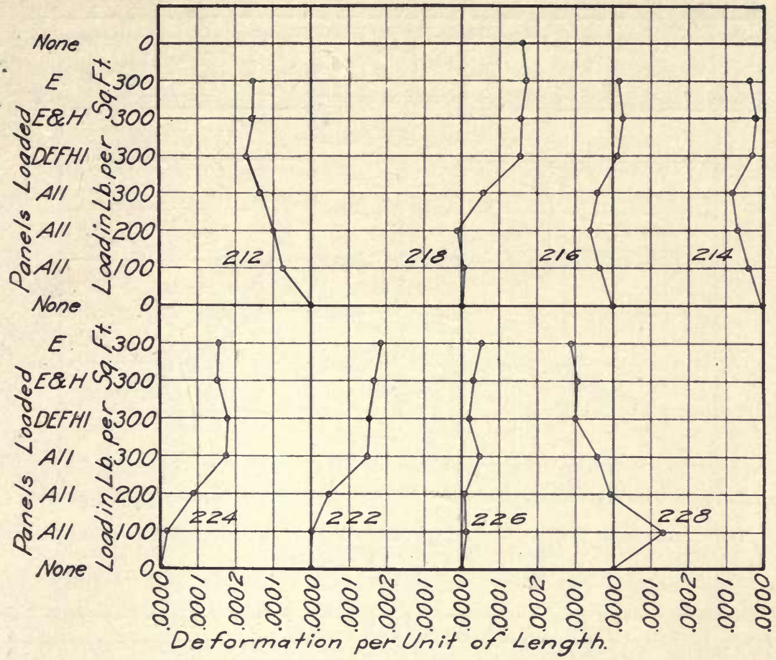

FIG. 57. LOAD-DEFORMATION DIAGRAMS FOR BENT-UP BARS AND STIRRUPS.

crete, on the assumptions made, is .077 for the middle of the beam and .07 for the end of the beam. These coefficients are lower than the value of 1/12 usually assumed in design of such beams.

36. *Girders.*—For the tensile stresses at the middle of the girders the observations showed about 8000 lb. per sq. in. in the reinforcement at the middle. This corresponds to a bending moment coefficient of .05, again neglecting the tensile strength of the concrete. The reinforcement at the end of the girder was inaccessible.

Assuming a modulus of elasticity of 2 500 000 lb. per sq. in., the concrete on the compressive side of the beam at the support showed a compressive stress of 900 lb. per sq. in. The reading at the middle of the beam showed very little compression. Assuming that the loads on the girder are concentrated at the points where the intermediate beams are connected, and making the same assumption of distribution of stress as before, the coefficient of bending moment was .06. It seems probable that the compression at the middle of the span must be distributed over a considerable width of floor, or larger readings of compression would have been obtained.

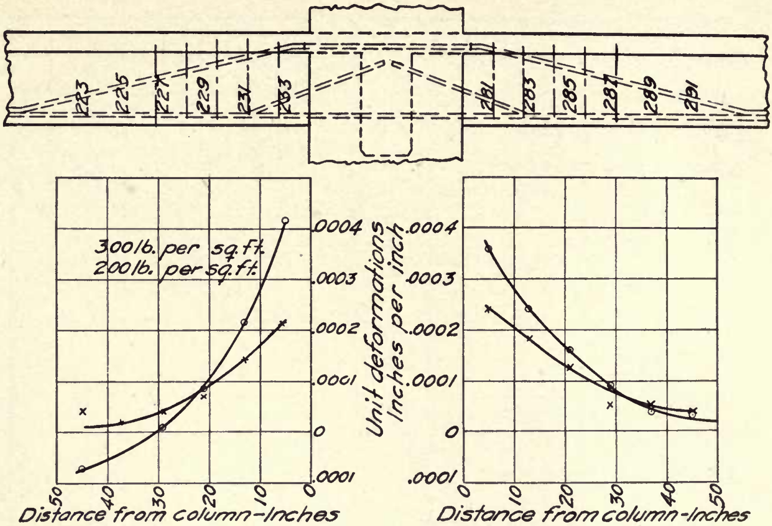

FIG. 58. DIAGRAM SHOWING DISTRIBUTION OF COMPRESSIVE DEFORMATION IN BOTTOM OF COLUMN BEAM.

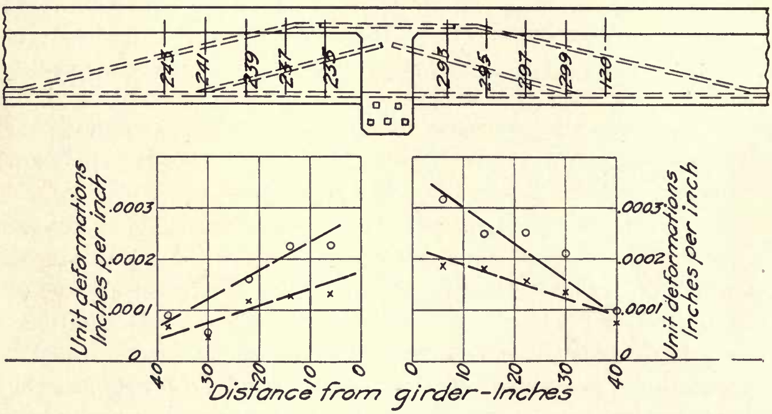

FIG. 59. DIAGRAM SHOWING DISTRIBUTION OF COMPRESSIVE DEFORMATION IN INTERMEDIATE BEAM.

37. *Decrease in Compression with Distance from Support.*—In four beams measurements of compressive deformations were taken at a series of gauge lines from the support to a location near the point of inflection. The position of these points is shown in Fig. 42. The gauge lines No. 223, 225, 227, 229, 231 and 233 are on one side of column No. 6, and 281, 283, 285, 287, 289 and 291 are on the other side of column

No. 6. It may be expected that there will be full restraint for the end of the beams. Gauge lines 243, 241, 239, 237 and 235 are on one side of a girder and 293, 295, 297, 299 and 1201 are on the other side. The unit-deformations for these gauge lines at loads of 200 lb. per sq. ft. and 300 lb. per sq. ft. are plotted in Fig. 58 and 59.

The measurements recorded for the column beams show considerably more compressive stress than do those for the intermediate beams, per-

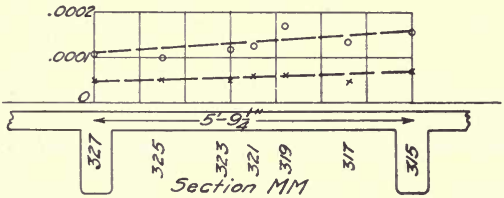

FIG. 60. DIAGRAM SHOWING DISTRIBUTION OF COMPRESSIVE DEFORMATION ACROSS FLANGE OF T-BEAMS.

haps one-third more. This difference in stress may be due partly to the deflection of the girder, and to the deflection of the intermediate beam between its support and a point opposite the end of the column beam, which would permit a larger part of the load to be carried by the column beam. It may be due somewhat to the fact that reinforcing bars are bent down from a point at the end of the column beam, while in the intermediate beams the bars run horizontally for a foot from the face of the girder.

The direction of the lines in Fig. 58 and Fig. 59 indicates a zero stress at about 45 in. from the face of column in the column beams and at about 50 in. from the face of the girder in the intermediate beams. In both cases the results locate the point of inflection at about 0.22 of the clear span.

38. *T-beam Action.*—The distribution of compressive stresses in the T-beam formed by a beam and the floor slab (which involves the distances away from the beam for which compressive stresses are developed) has been a fruitful source of discussion. Measurements parallel to the axis of the beam were taken on the upper surface of the floor slab immediately above beams and at intervals between them. These gauge lines are No. 315, 317, 319, 321, 323, 325 and 327 (see Fig. 42). The deformations are shown in Fig. 52 and 56. The amount of these deformations at points across the slab for loads of 200 lb. and 300 lb. per sq. ft. is shown in Fig. 60. It is apparent that a somewhat higher stress existed in one beam than in the other. Taking this into consideration,

the compressive stress varies quite uniformly from one beam to the other, and the full width of the floor slab may be said to be effective in taking compression. The overhang (counting to the mid-point between beams) is 6½ times the thickness of slab. It will be noticed that the conclusions are the same as given for the Wenalden building test.

Readings were also taken on the under side of the floor slabs parallel to the beams at three places (No. 1205, 1211 and 1213), but the conditions attending the location of these points do not permit conclusions to be drawn.

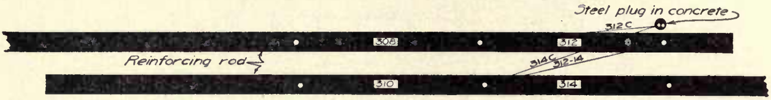

FIG. 61. ARRANGEMENT OF GAUGE LINES TO TEST FOR MOVEMENT OF BAR RELATIVE TO CONCRETE.

39. *Floor Slab.*—Measurements were taken on the floor slab in the direction of its span at three places on the under side and at one place on the upper side immediately above one of the lower measurements. These gauge lines were No. 277 on the under side of the slab close to a girder (Fig. 42), No. 279 on the under side of the slab 5 ft. from the edge of the girder, No. 309 (Fig. 43) on the upper surface immediately above No. 279, and No. 1203 (Fig. 42) on the under side half way between girders. The measurements are plotted in Fig. 55 and 56. As might be expected from being close to the girder and near the level of its neutral axis, No. 277 showed little deformation. The pair of gauge lines (No. 279 and 309) shows less deformation than would be calculated by the ordinary beam formula, but perhaps not less than would be the case if the tensile strength of the concrete is considered to be quite effective. The reading of No. 1203 was even smaller than No. 279. All the stresses found in the floor slab were low. The deformations parallel to the beams were discussed under T-beams.

40. *Bond Stresses.*—At the ends of the beams the reinforcing bars lapped over the center line of the girder a distance of 15 in. An effort was made to determine whether there was a movement of one of these bars with reference to the adjoining concrete and with reference to the adjoining bar; also whether the deformation in the stub end of the reinforcing bar was the same as in the adjoining bar. Fig. 61 shows the location of the reinforcing bars with reference to each other, and the position of the gauge lines. No. 312-14 in comparison with No. 312 and 314 will indicate any relative movement of one bar with respect to

the other, and No. 312c and 314c in comparison with No. 312 and 314, respectively, will indicate any movement of the bars with respect to the concrete.

It appears possible that the initial reading of No. 314 is slightly in error, and the remarks already made about quantitative interpretation of results and the chances for variations in stresses in adjacent bars or in adjoining concrete should be borne in mind in studying the results. It seems evident that No. 314 (on the lapped bar) records considerably less stress than (Fig. 50, p. 74) No. 312. The measurements indicate a possibility that the right-hand point of gauge line No. 314 has moved to the right relatively to the right-hand point of No. 312, though this amount may not be more than the amount of initial slip necessary to develop the requisite bond stress. The measurements taken have no bearing on whether the left-hand point of No. 314 has moved. The measurements also indicate that there was no motion of the left-hand point on the reinforcing bar (No. 312 gauge line) relatively to the concrete at its side, though it must be borne in mind that the point taken was so close to the bar that only slip and not distortion of concrete could be measured.

41. *Web Deformations.*—No diagonal tension cracks were visible on any of the beams or girders.

In girder 4 measurements were taken on the diagonal portion of a reinforcing bar, one of the bars which is provided to take negative bending moment. This is shown in Fig. 42, Section K-K. The gauge lines are No. 222, 224 and 226. The position of the gauge lines is also shown in Fig. 41. The measurements are plotted in Fig. 57. It was impracticable to measure the deformation at a point closer to the support. The measurements show about the same stress at No. 222 and 224, perhaps 5000 lb. per sq. in. The stress at No. 226 is materially less. It is not improbable that there was tension in this rod throughout its length. As there was considerable compression measured in the gauge lines on the bottom of the girder below No. 222, it seems probable that a crack was formed in the top of the floor slab somewhere above No. 222, but as this space was filled in with bags of cement no observation was made during the test, and inspection of this space after the load was removed seems to have been overlooked. At the other end of the girder, near column 6, a fine test crack was found on the upper surface of the floor 2 in. from the face of the column extending across the width of the girder and beyond. This extended through the floor. A similar crack was observed on girder 3 near column 15.

Gauge line No. 228 is on a stirrup (see Fig. 41). This stirrup is in an inclined position. It is not known what bar it is intended to be connected with, nor whether there is connection with a tension bar. The gauge line is in a region of the beam where horizontal compressive stresses may be expected. The measurement in the stirrup at the first increment of load shows tension (see Fig. 57) and subsequent increments give compression. It should be noted that readings could not be taken on the upper end of the stirrup. If the upper ends are merely bent out into the floor slab it is hard to see that the stirrup may be expected to be useful in transmitting web stresses.

In beam 9 (see Fig. 42, section L-L, gauge line No. 218) measurement was taken on the diagonal portion of a reinforcing bar which is carried through the girder at its top and a few inches beyond. See also Fig. 41. This shows a tension of 3000 to 5000 lb. per sq. in. (See Fig. 57.) This bar was inaccessible from the top of the floor, but the gauge lines on the companion bar (No. 324 and 318) show about 5000 and 9000 lb. per sq. in. Measurements in the diagonal portion of a single-bend bar (gauge lines No. 216 and 214, Fig. 42) which extends only to the center of the supporting girder indicate a small compression in the bar (see Fig. 57). A stirrup, which like the one in the girder was close to the end of the beam and was inclined so that its lower end was nearer the support than its upper, showed shortening of the stirrup (see gauge line No. 212, Fig. 41, 42 and 57). In both cases, the arrangement was such that the stirrup could hardly be effective.

The amount of the vertical shear in the beams and girders was such that diagonal tension cracks might be expected except for the small tensile stresses in the top of the girder and the end constraint which seems to have been developed in both beams and girders.

42. *Deflections.*—The deflections of the beams (including that due to deflection of girder) and the deflections of girders are given in Fig. 62. The location of the deflection points is shown in Fig. 44. The effect of time upon the deflection is shown by the increase in deflection under constant load. The change when portions of the load had been removed may be due in part to the time element and in part to the effect of location of the load on the panels. The deflections seem relatively small, especially when compared with deflections obtained in laboratory tests of beams carrying the same loads. The conditions were such that the supports were subject to possible displacement by workmen.

43. *Effect of Number of Panels Loaded.*—In taking off the load, the outer panels were unloaded first, and observations were taken on the remaining panels in an attempt to determine the relation between single panel loading and group loading. Panels B and C were first unloaded (see Fig. 42), then panels D and F, then panels H and I, and finally

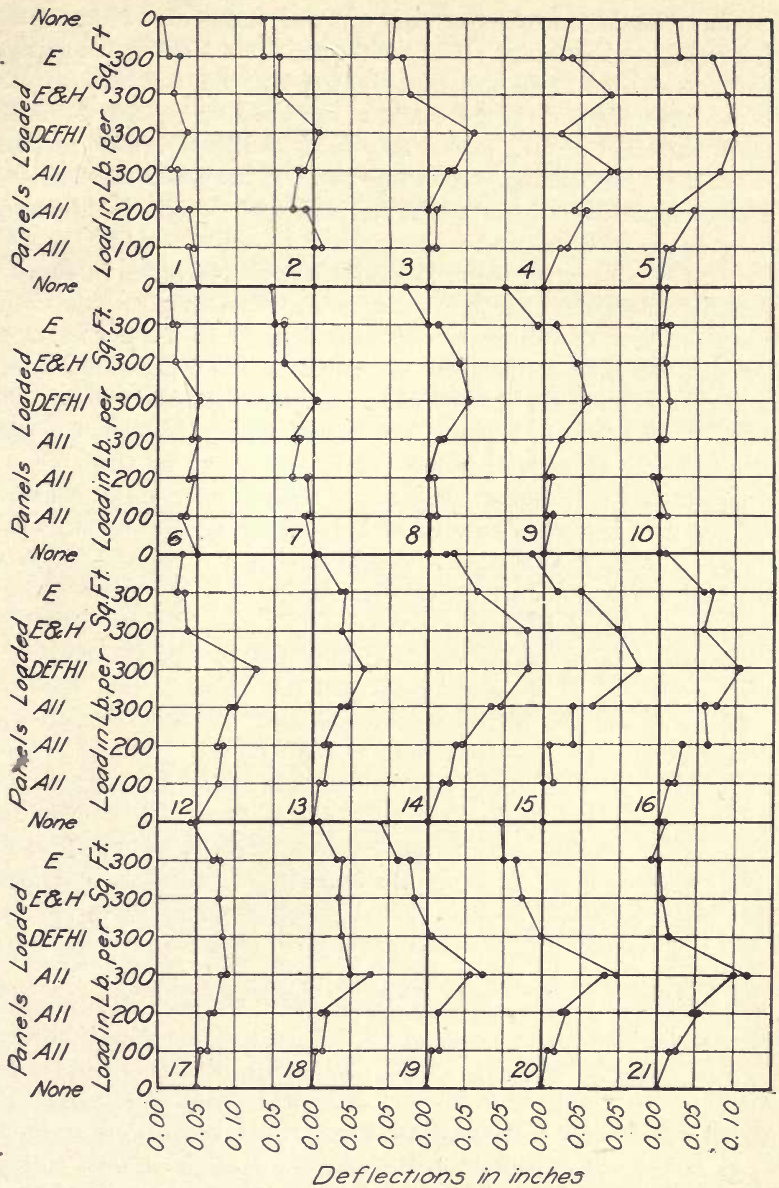

FIG. 62. LOAD-DEFLECTION DIAGRAMS.

panel E. The deformations at each of these stages are shown in the load-deformation diagrams. If at each stage of the loading the average of the deformations at all the points having a similar location (say the points on the under side of the beams at the south end of the test area) be taken the effect of area loaded may be judged by the ratios of these values to the corresponding ones at full load. If the beams be considered as freely supported (without restraint) and their weight be neglected, and if it be assumed that no time is required for adjustment of members to the load coming upon them, it should be possible in many cases to forecast the effect of a change in the area loaded. Comparing the ratios referred to above (no diagrams reproduced here) with what might be expected on the basis of the above assumptions, it is found that in most cases the direction of the changes in stress agrees with predictions. The amount of change to be expected can not be predicted because of complications in the division of load between elements of the structure. A point worthy of note is that the stresses at the center of the panel E are 30 per cent less when only panel E is loaded than when the whole test area is loaded. This must be due to the fact that with the removal of the load which rested on the side panels F and D the column beams at the edge of panel E recover a considerable part of their deflection, and because of their smaller deflection they will receive the effect of a greater proportion of the panel load than taken before, thus relieving the interior beams somewhat. The stresses were decreased at this stage more than they were increased later by the removal of the load in the end panel H. This indicates that the stiffness of the floor system permits considerable lateral distribution of the load-carrying stress. The removal of the load in panel H increased the stress in the beams at the center of panel E much as though the beams were continuous and freely supported. This would indicate that the most severe condition of loading affecting the center of the beams is brought about by loading several panels which lie side by side and are not separated by girders.

44. *Effect of Time on Stresses Developed.*—To determine the effect of time-under-load on the amount of deformation developed, observations were taken at each stage of the loading after the load had been in position for from 8 to 12 hours and also at intervals of 8 to 16 hours when all panels were fully loaded. The latter investigation continued over 48 hours. The results found during the loading seem to indicate in a general way a tendency for the deformations at the ends of beams both above and below to increase and also those at the centers of the beams above, but on the lower surface of the beams at the center the tendency was to decrease. No reason is apparent why the changes on the

compression and tension surfaces at the center of the beam should be in opposite directions and it is probable that this result is erratic. With full load the time effect at only a few gauge lines was observed; only two of them were at the center of the beams, both being below and none above. These measurements also indicate an increase in deformation at the ends of beams both above and below. At the center of beam below, one gauge line shows an increase and the other a decrease in deformation thus giving no results.

45. *Columns.*—Readings were taken on the four faces of column No. 5 just below the girders, but the results are not consistent enough to warrant attempting to draw conclusions.

46. *Test Cracks.*—Fine tension cracks were observed in the lower part of the beams and girders. The location of the observed cracks is shown on Fig. 63. The appearance of these fine cracks is similar to those

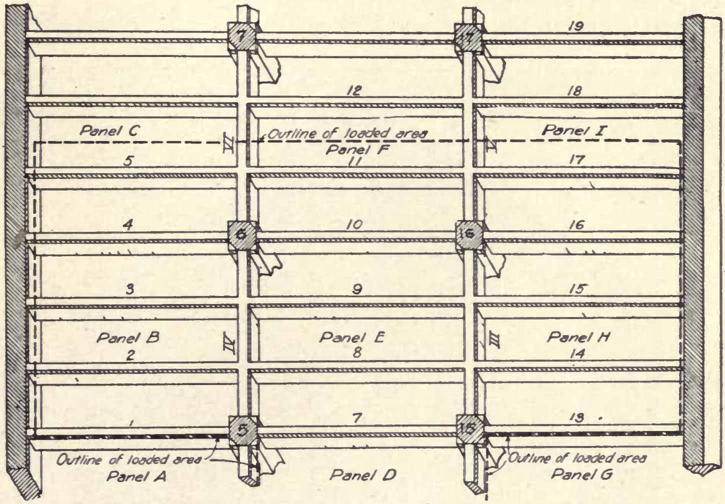

FIG. 63. CABINET PROJECTION SHOWING BEAMS AND GIRDERS AND POSITION OF TEST CRACKS.

observed in laboratory tests. They would not be noticed without specially careful examination.

The floor cracks already mentioned indicate the development of the tensile stresses in the beams and girders at the support.

It was not possible to give full attention to every feature upon which information was sought, and in some cases isolated points were used with a view of determining tendencies, and in these naturally there is less certainty in the indications.

V. The Deere and Webber Building Test.

47. *The Building.*—The Deere and Webber Building is an eleven-story and basement warehouse at Minneapolis, Minnesota, owned by the Deere and Webber Company. It was built by the Leonard Construction Company of Chicago. Fig. 64 is a view of the building at the time of test. Fig. 65 shows the floor plan of the building and the location of the panels loaded. The dimensions of the panels are 18 ft. 8 in. by 19 ft. 1 in. A 1-2-4 mixture was used, the slab thickness measuring 9 3/16 in. The floor was designed by the Concrete Steel Products Company for a live load of 225 lb. per sq. ft., and the details of the reinforcement are shown in Fig. 66. The floor tested was the fourth from the ground and the conditions were not such as to make a high showing of strength. Owing to a failure in the supply of aggregates during the construction of this floor, an abnormal number of bulkhead separations occur in the slab, as is shown in Fig. 65. Such separations

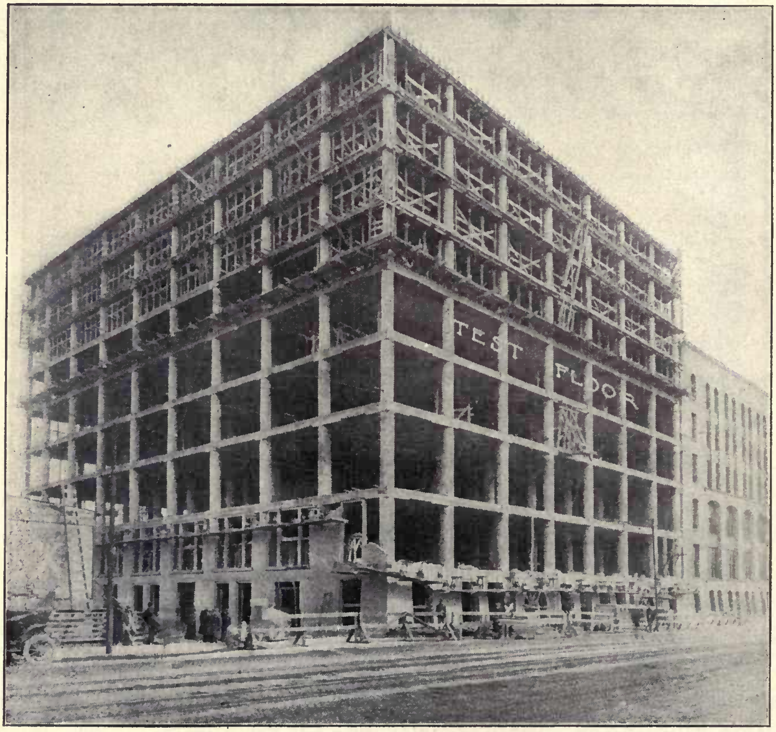

Fig. 64. Deere and Webber Building at the Time of Test.

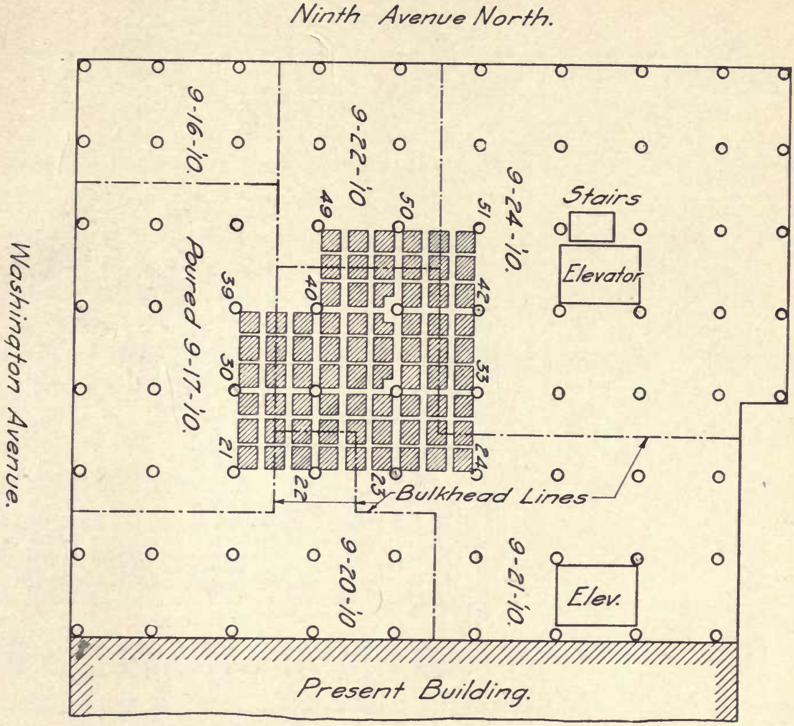

FIG. 65. PLAN OF FLOOR SHOWING LOCATION OF PANELS TESTED.

occur in every panel under load except one. The concrete was only 40 days old at the beginning of the test. In general the conditions were such as to give slightly higher stresses than would be expected had the slabs been well seasoned and normally poured.

48. *Method of Testing.*—Fig. 66 and Table 10 show the position of points at which measurements of deformation were made. The numbers given are those used in recording and plotting the data in the tables and diagrams. The total number of readings was in excess of 3300. The falsework for instruments and observers is shown in Fig. 67. For measuring deflections the instrument shown in Fig. 68 was used. A polished steel ball was attached to the ceiling, another was carried on an upright, and the instrument was inserted between them. Measurements were made in this manner to the nearest .001 inch with accuracy. For measuring the deformation in the reinforcement at the center of the span a clamp was rigidly attached to the slab rod (the concrete being removed at one point for this purpose), and a Wissler dial was carried

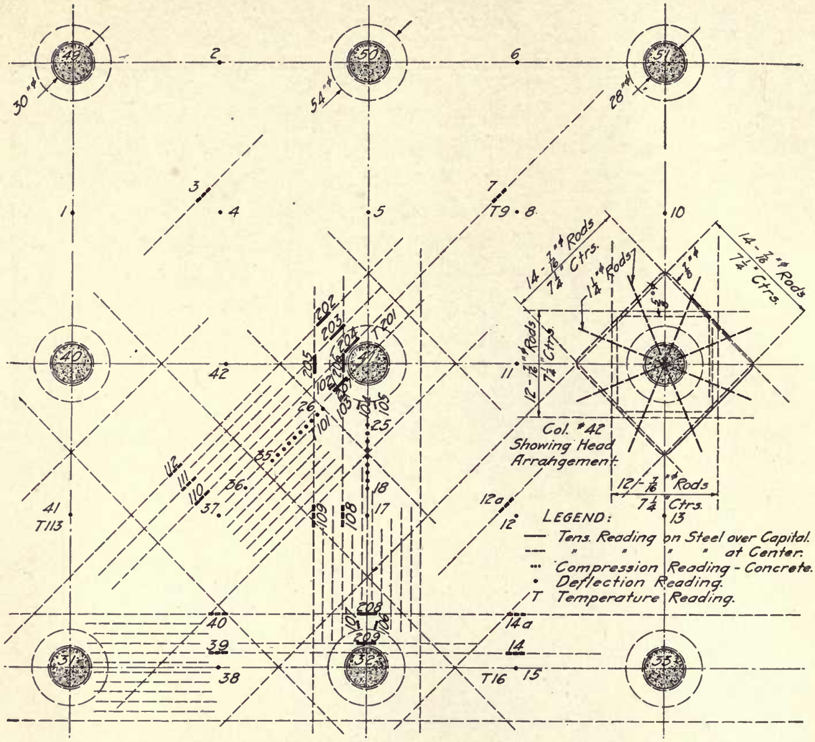

FIG. 66. ARRANGEMENT OF REINFORCEMENT AND LOCATION OF OBSERVATION POINTS.

on the clamp (Fig. 69). A fine silk-covered copper wire was attached to the rod at a distance of 15 in. from the clamp and passed immediately below the rod, over an idler on the clamp, and then over the drum of the dial. As this wire was 1/16 in. below the under surface of the slab rod, the deformations observed were only slightly in excess of the deformation in the rod. The wire was placed in this position because experience in the laboratory has demonstrated that measurements taken below the slab depending upon the position of the neutral axis for correction, are subject to considerable error. By this arrangement the deformation was measured to an indicated .0002 in. on a gauge length of 15 in. The measurement was less responsive to slight changes than were the other measurements made.

For measurements of deformation in the reinforcement over the column capital the University of Illinois type of Berry extensometer built for this test (Article 11) was used. A gauge length of 15 in. was

TABLE 10.

DATA ON POSITION OF RODS ON WHICH DEFORMATIONS WERE MEASURED.

Gauge Line	Band	Position in Band	Embedment inches*		Layer of Steel over Column
3	Diagonal	3d rod from center	....	$\frac{11}{16}$	
7	"	" " " "	$\frac{15}{16}$	....	
12a	"	2d " " "	....	$\frac{3}{8}$	
14	Cross	" " " "	$\frac{5}{16}$	....	
14a	"	Outer rod of band	....	$\frac{1}{2}$	
39	"	2d rod from center	$\frac{3}{8}$	....	
40	"	Outer rod of band	....	$\frac{11}{16}$	
108	"	3d rod from center	$\frac{15}{16}$	....	
109	"	Outer rod of band	....	$\frac{13}{16}$	
110	Diagonal	3d rod from center	$\frac{3}{4}$	....	
111	"	5th rod from center	....	$\frac{13}{16}$	
112	"	Outer rod of band	$\frac{9}{16}$	....	
202	"	" " " "	....	$2\frac{3}{4}$	2d layer from top
203	"	5th rod from center	$2\frac{1}{16}$	....	" " " "
204	"	3d " " "	....	$1\frac{7}{8}$	" " " "
207	"	1st " " "	$2\frac{3}{8}$	....	" " " "
205	Cross	Outer rod of band	....	$3\frac{1}{4}$	3d " " "
206	"	3d rod from center	3	....	" " " "
208	"	Outer rod of band	....	$2\frac{3}{4}$	" " " "
209	"	3d rod from center	$2\frac{1}{8}$	....	" " " "

*Measurement from surface to center of rod.

used. For measuring deformations in the concrete the original Berry 6-in. extensometer (Fig. 13) was used.

49. *Loading and Testing.*—In applying the load care was taken that no serious arch action in the load be possible. In the earlier stages brick were piled in piers, as shown in Fig. 65 and in Fig. 70, with open aisles from 8 in. to 16 in. wide between the piers. For the later loading, cement in bags was used as loading material, the piers being kept separate as before. The load given in the tables is in all cases the total load on the panel divided by the area of the panel, the intensity of the load under the pier being greater. The aisles gave an opportunity for making observations upon the concrete and reinforcement.

To correct for temperature variations one entire day was spent in observing effects due to temperature alone, and the large Berry extensometer was read on a standard bar before and after each series of slab readings.

The test continued for six days from October 30 to November 4, inclusive, 1910. Eight panels were loaded. First readings were taken on all instruments with the floor unloaded and then a load equal to 75 lb. per sq. ft. was applied over the entire eight panels. Another series of observations was taken and the load increased to 150 lb. per sq. ft. In this manner alternate observations and loadings were continued for three

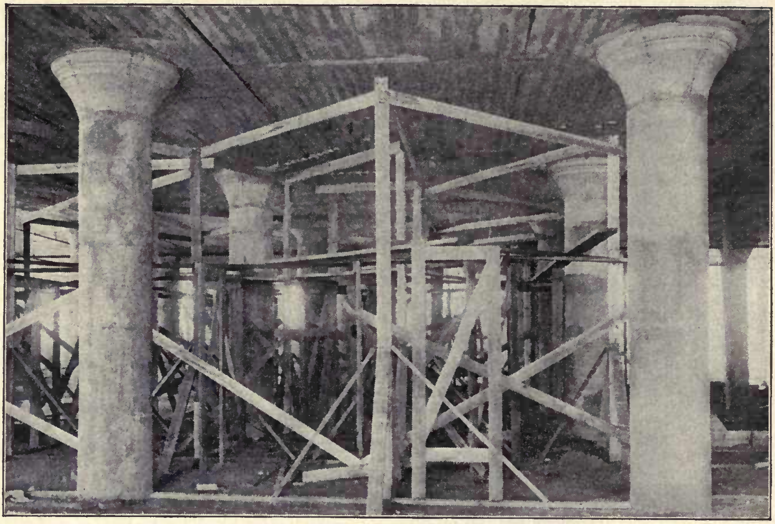

FIG. 67. FALSE WORK FOR INSTRUMENTS AND OBSERVERS.

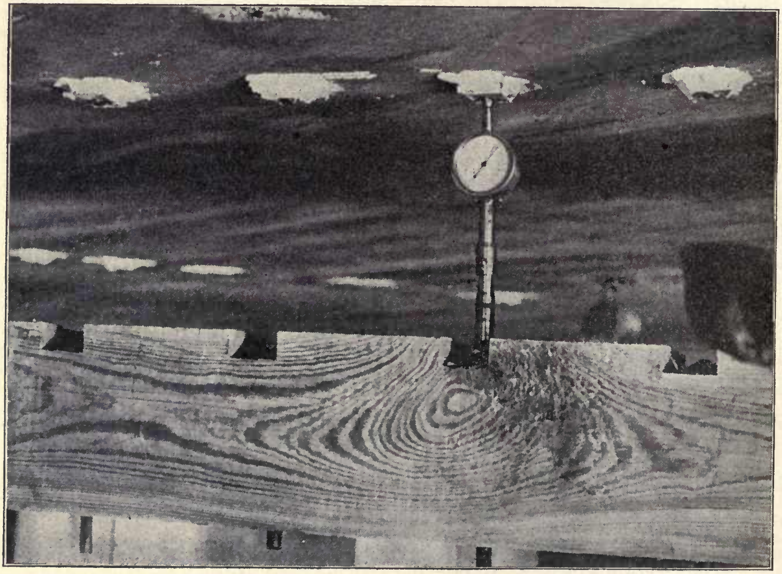

FIG. 68. DEFLECTOMETER IN PLACE.

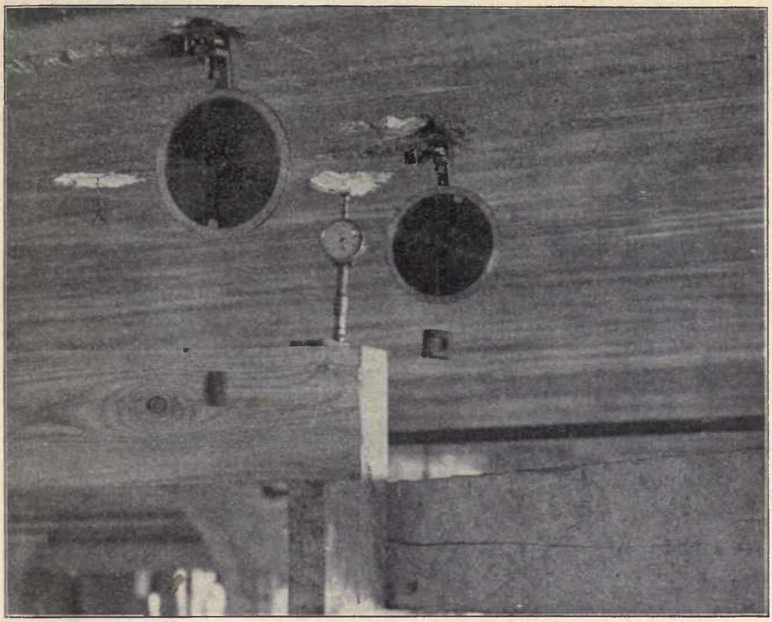

FIG. 69. WISSLER DIAL FOR MEASURING DEFORMATION IN REINFORCEMENT.

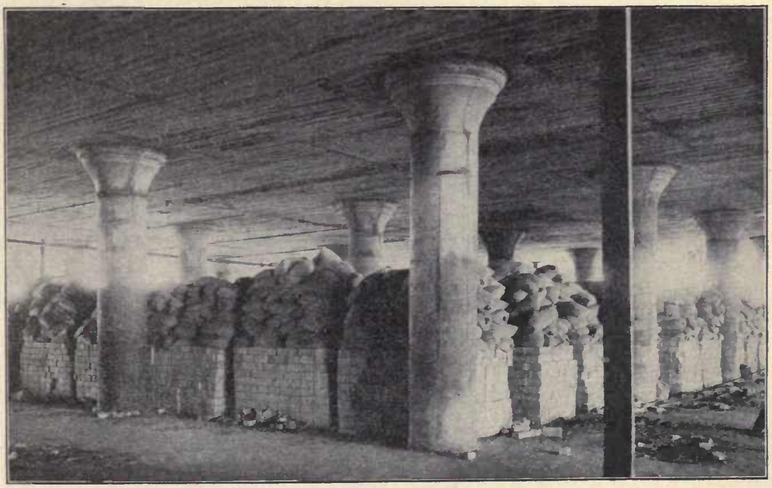

FIG. 70. VIEW OF MAXIMUM TEST LOAD.

TABLE 11.

Deflection of Slab in Inches at Points Midway between Columns.

Load in lb. per sq. ft.		Observation Point Number															
Center Panel	Outer Panels	37	4	8	12	17	42	5	11	15	38	41	1	2	6	10	13
0	0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
75	75	29	29	32	16	16	13	18	18	22	20	24	7	11	11	15	10
150	150	61	73	79	66	39	30	57	45	44	38	49	22	22	17	27	17
187.5	187.5	74	101	111	95	52	35	81	63	58	57	64	33	26	19	34	20
187.5	187.5	0.085	0.121	0.123	0.105	0.063	0.049	0.089	0.070	0.072	0.076	0.077	0.046	0.041	0.041	0.050	0.038
225	225	98	139	143	119	74	61	107	82	83	83	86	52	45	43	56	45
262.5	262.5	124	173	180	149	89	75	138	103	103	93	111	64	48	45	69	50
300	300	168	225	242	186	105	101	183	128	126	93	140	78	48	57	88	57
300	300	177	234	252	193	110	109	189	132	135	99	148	80	52	62	96	62
300	300	0.180	0.242	0.258	0.198	0.113	0.112	0.193	0.134	0.139	0.106	0.152	0.087	0.059	0.068	0.101	0.064
350	350	224	272	293	231	133	132	218	157	156	121	178	97	70	77	117	76
350	350	217	281	301	235	135	131	224	161	161	123	182	102	71	79	121	81
350	350	220	285	306	240	137	138	227	163	162	125	186	101	73	79	122	80
350	350	223	288	309	242	138	143	229	163	162	125	186	102	73	80	121	80
350	350	224	....	....	244	139	137	231	165	163	126	188	102	73	81	123	81
350	350	227	....	....	246	141	140	235	167	165	128	191	103	74	82	125	83
350	350	0.234	0.299	0.321	0.250	0.146	0.146	0.238	0.169	0.170	0.134	0.196	0.107	0.078	0.088	0.130	0.090
350	187.5	246	223	241	188	138	134	172	124	125	127	181	80	63	70	104	73
350	0	266	115	114	97	124	112	72	50	54	115	156	34	58	41	58	43
350	0	274	116	111	96	126	117	68	47	54	122	160	36	42	46	61	48

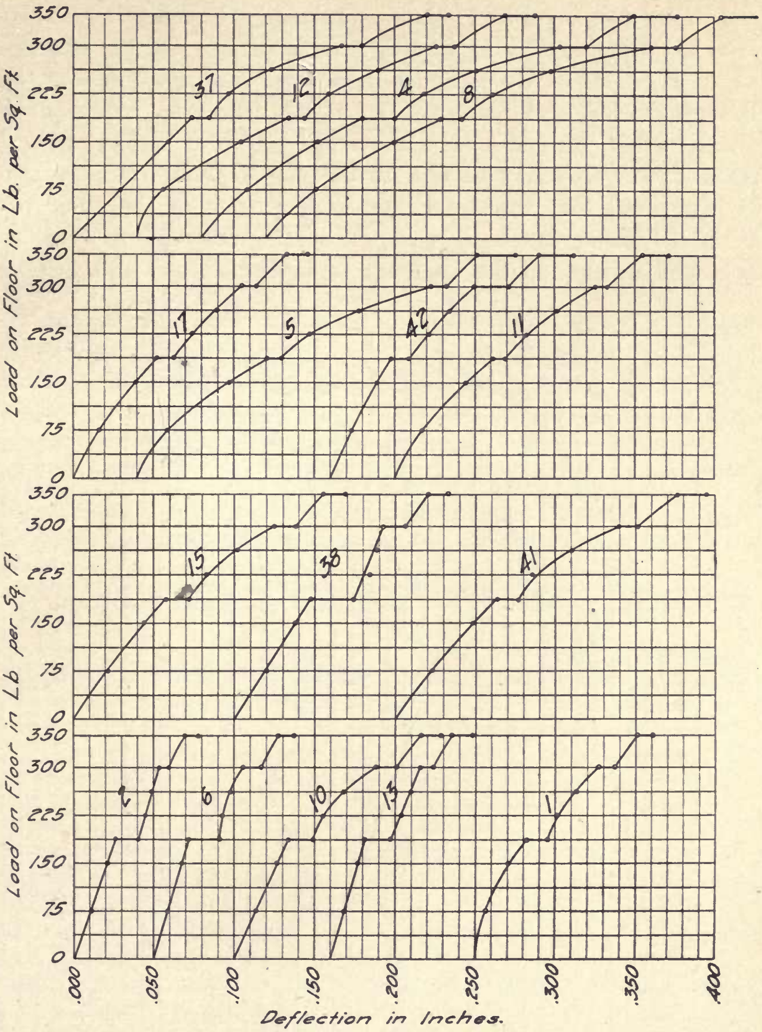

FIG. 71. DIAGRAM OF DEFLECTIONS.

days. Over-night readings were taken on one or two occasions in the evening, about midnight and in the morning. The maximum load of 350 lb. per sq. ft. was allowed to remain on the floor about 22 hours, readings being taken at frequent intervals during that time. In the process of unloading the outer panels were first cleared, and finally the load was removed from the center panel. Readings were taken at intervals during the progress of the unloading. The data obtained are presented in Tables 10-15 and plotted in Fig. 71-74.

50. *Deflections.*—Fig. 71 shows graphically the deflections at sixteen points, the same data being recorded in Table 11. On the second diagram of Fig. 71 note the comparison between readings 5 and 11, where bulkheads existed, and readings 17 and 42, where no bulkheads were present. Other instances of the marked effect of bulkheads on the stiffness of the slab may be seen in the plotted data. It may also be said in general that the deflections were greater in the outer panels than in the center panels, in part due to the bulkheads in these outer panels, and in part to the tendency to higher stresses and deflections

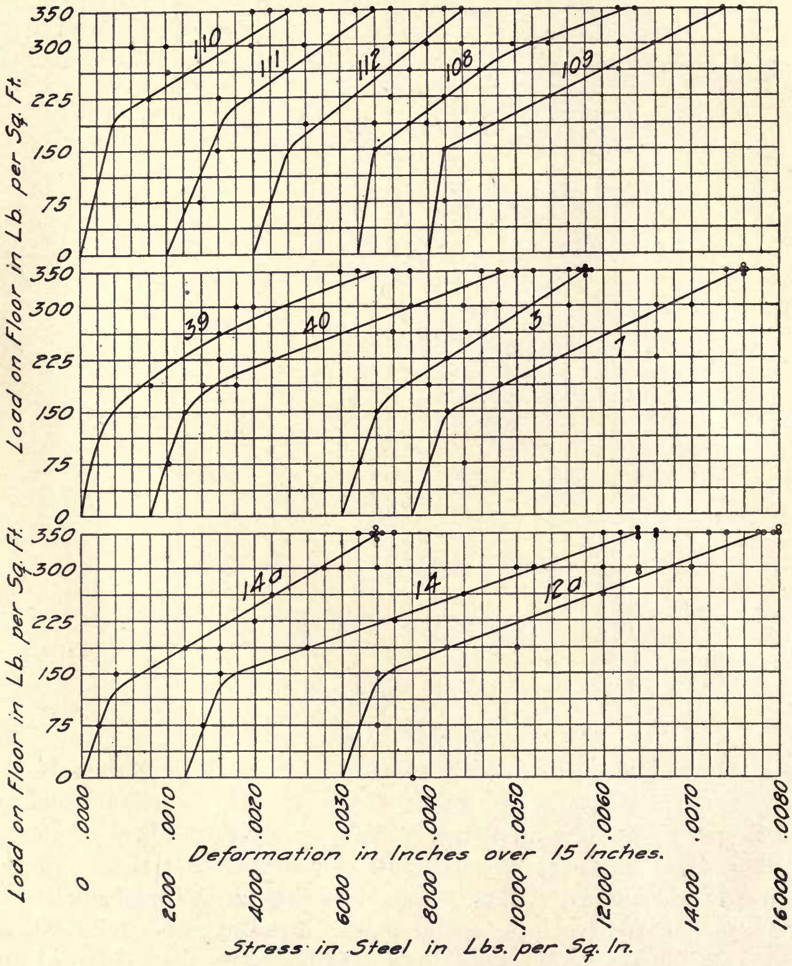

FIG. 72. DIAGRAM SHOWING STRESS IN REINFORCEMENT AT CENTER OF SPAN.

in end panels. The deflections probably would have been smaller with well cured concrete and in considering deflections it must be remembered that this slab was only 43 days old when the maximum load was placed upon it. The maximum deflection found was .32 in., which is 1/1000 of the span. This was at a bulkhead in an outer panel. In the center panel the deflection for all eight panels loaded, was .227 in., or 1/1400 of the span, which increased to .274 in. or 1/1200 of the span, when the load was removed from the outer panels.

51. *Stress in Reinforcement at Center.*—Fig. 72 and Table 12 give the data on the measured deformations in the reinforcement at the center of the spans. The table is reduced to unit deformations while the diagrams show total deformation over the lengths gauged. The stresses observed at the center were very low. On the upper diagram in Fig. 72 are shown deformations in the center panel and it is to be noted that these are, in general, smaller than those in the outer panels. This would seem to indicate that the reinforcement at the center of the span should be designed for one panel loaded, as this apparently gives a worse condition at the center than full loading. The observed stresses indicate that the diagonal and cross band rods took practically the same stress.

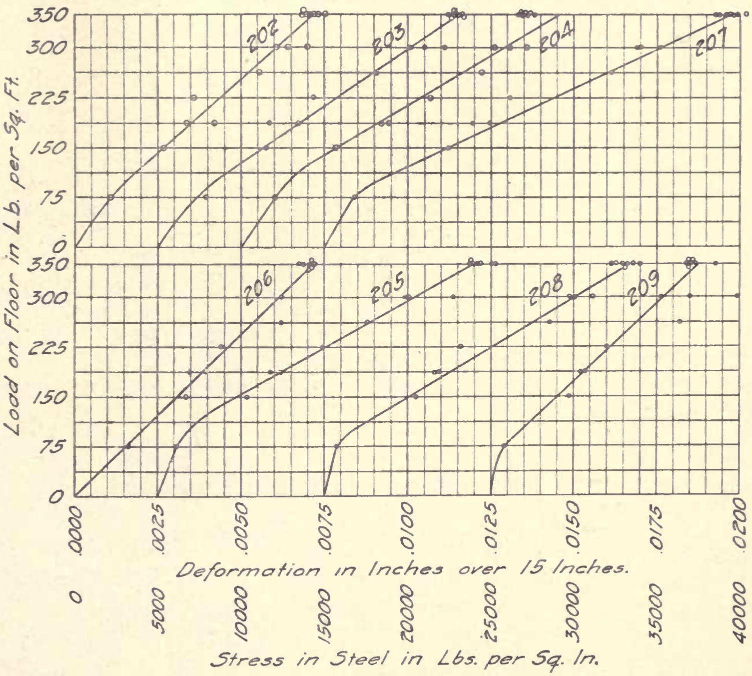

FIG. 73. DIAGRAM SHOWING STRESS IN REINFORCEMENT OVER CAPITAL.

TABLE 12.

Unit-deformation in Reinforcement at Center of Span between Columns.

Load in lb. per sq. ft.		Gauge Line											
Center Panel	Outer Panels	39	40	108	109	14	14a	110	111	112	7	3	12a
0	0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
75	75	– 1	1	– 3	1	0	1	– 5	3	– 3	– 1	1	– 3
150	150	– 1	3	1	1	1	3	– 5	4	– 4	– 3	3	– 3
187.5	187.5	0	4	4	3	8	8	– 3	– 1	1	1	7	3
187.5	187.5	0.00005	0.00007	0.00005	0.00004	0.00013	0.00011	–.00001	–.00001	0.00007	0.00007	0.00007	0.00008
225	225	11	9	7	9	15	13	5	4	8	12	8	1
262.5	262.5	11	19	9	15	20	15	7	9	9	12	9	15
300	300	21	20	12	15	24	19	7	3	11	12	12	17
300	300	13	20	12	13	25	20	4	3	8	15	12	17
300	300	0.00021	0.00024	0.00015	0.00017	0.00031	0.00023	0.00013	0.00011	0.00013	0.00021	0.00017	0.00021
350	350	21	27	20	24	31	24	15	17	13	21	19	23
350	350	20	29	20	24	32	23	15	15	12	20	19	24
350	350	21	25	21	24	33	23	13	16	13	20	19	28
350	350	24	28	20	23	35	23	16	16	12	20	19	27
350	350	25	27	21	23	33	23	19	16	11	19	17	28
350	350	21	27	20	24	33	23	17	12	9	19	19	28
350	350	0.00020	0.00025	0.00021	0.00024	0.00035	0.00023	0.00016	0.00011	0.00009	0.00020	0.00019	0.00027
350	187.5	19	25	23	26	31	17	19	9	12	17	17	19
350	0	17	28	21	26	11	– 1	21	15	15	9	4	3

52. *Stress in Reinforcement at Column Capital.*—Table 13 and Fig. 73 give the data on the stress in the reinforcement over the column capital. The upper diagram covers diagonal rods, the lower diagram cross band rods. Among the diagonal rods it may be noted that the stress in rod No. 207 was measured over the edge of the capital while that in No. 203 and 204 was measured opposite the center of the column. The higher stress in No. 207 would seem to indicate that the stress in these rods decreases, passing from the critical section at the

TABLE 13.

Unit-deformation in Reinforcement over Capital.

Load in lb. per sq. ft.		Gauge Line							
Center Panel	Outer Panels	202	203	204	207	205	206	208	209
0	0	.00000	.00000	.00000	.00000	.00000	.00000	.00000	.00000
75	75	14	8	7	6	4	11	2	2
150	150	24	22	19	25	18	22	19	14
187.5	187.5	29	28	30	33	24	26	23	18
187.5	187.5	.00035	.00022	.00028	.00030	.00022	.00023	.00022	.00018
225	225	30	31	38	37	26	29	28	23
262.5	262.5	44	44	48	58	42	42	45	52
300	300	47	50	50	63	50	42	50	50
300	300	49	54	54	63	50	42	49	34
300	300	.00053	.00058	.00057	.00067	.00059	.00047	.00054	.00040
350	350	52	60	58	72	62	48	57	39
350	350	54	59	56	73	62	47	61	45
350	350	55	60	58	75	64	47	60	40
350	350	53	58	57	75	62	45	58	40
350	350	52	60	57	72	68	47	59	40
350	350	54	61	56	76	67	48	61	41
350	350	.00056	.00061	.00056	.00078	.00064	.00049	.00063	.00041
350	187.5	53	55	54	80	64	48	60	41
350	0	50	48	51	77	62	48	55	39

edge of the capital to any section nearer the center of the column. This is as would be expected. The stresses found from these readings indicate clearly that the slab should be designed for a maximum moment over the support and not at the center. In the design of this building some 75% more reinforcement was provided over the support than was used at the center.

53. *Stress in Concrete at Edge of Capital.*—Fig. 74 and Table 14 give deformations observed in the concrete at the edge of the column capital. Owing to the fact that when the slab was poured there was no intention of testing, no specimens of the concrete were available from which to determine the modulus of elasticity. Hence it is necessary to assume a value for concrete about 40 to 45 days old cured in fall weather at Minneapolis. From experiments made at the University of

TABLE 14.

UNIT-DEFORMATION IN CONCRETE AT EDGE OF CAPITAL.

Load in lb. per sq. ft.		Gauge Line					
Center Panel	Outer Panels	102	103	104	105	106	107
0	0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
75	75	11	17	9	7	8	7
150	150	21	28	21	22	17	20
187.5	187.5	24	30	24	16	18	20
187.5	187.5	0.00015	0.00021	0.00018	0.00013	0.00015	0.00012
225	225	15	25	20	17	14	20
262.5	262.5	28	37	31	28	25	31
300	300	36	43	38	36	32	34
300	300	39	44	38	33	30	33
300	300	0.00032	0.00038	0.00033	0.00029	0.00024	0.00031
350	350	41	47	40	38	32	38
350	350	43	47	42	39	33	38
350	350	43	49	43	42	35	39
350	350	44	45	44	42	34	37
350	350	47	48	46	42	34	40
350	350	49	50	48	41	36	40
350	350	0.00047	0.00045	0.00046	0.00040	0.00034	0.00040
350	187.5	47	49	44	36	31	37
350	0	44	44	38	23	22	30

Illinois concrete of the same age cured under laboratory conditions showed a modulus of 1875000 lb. per sq. in., and in Table 15 this modulus has been used as giving the value for the concrete stress. In Fig. 74 a stress of 100 lb. per sq. in. corresponds to a deformation of .00032 in. if a modulus of 1875000 be assumed.

An interesting feature shown in the curves is the falling off in the concrete deformation when the load was allowed to remain over night. The decrease is less marked at higher loads than at low loads,

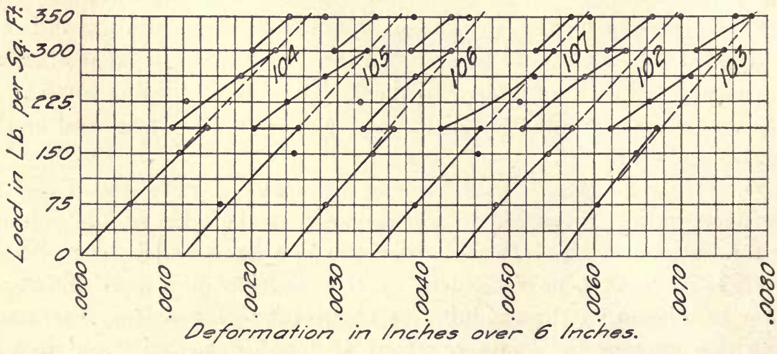

FIG. 74. DIAGRAM SHOWING STRESS IN CONCRETE AT EDGE OF CAPITAL.

while readings taken at very frequent intervals while the maximum load was on the floor showed that at first the stress steadily increased and the decrease did not begin until some time after the load was applied. The phenomenon is of interest as showing the readjustment in stresses which takes place under load even in the least plastic constructions. In general the concrete stresses checked those found in the reinforcement over the support.

54. *Summary of Stresses.*—Table 15 gives a summary of the stresses found at various points under the design load of 225 lb. per sq. ft. and also under the maximum load applied of 350 lb. per sq. ft.

TABLE 15.

Summary of Stresses.

Stresses are given in pounds per square inch.

		Design Load—225 lb. per sq. ft.			Maximum Load—350 lb. per sq. ft.		
		L. L.	D. L.	Total	L. L.	D. L.	Total
Reinforcement Over Head:							
Diagonal Band.	Maximum..	13 800	6 900	20 700	24 200	6 900	31 100
	Average....	11 000	5 500	16 500	18 800	5 500	24 300
Cross Band.	Maximum..	10 000	5 000	15 000	18 800	5 000	23 800
	Average....	9 000	4 500	13 500	17 200	4 500	21 700
Reinforcement at Center:							
Diagonal Band.	Maximum..	2 400	1 200	3 600	4 800	1 200	6 000
	Average...	2 000	1 000	3 000	4 800	1 000	5 800
Cross Band.	Maximum..	2 800	1 400	4 200	8 000	1 400	9 400
	Average....	2 500	1 300	3 800	6 600	1 300	7 900
Outer Panels.	Maximum..	4 600	2 300	6 900	10 400	2 300	12 700
	Average....	3 800	1 900	5 700	8 000	1 900	9 900
Concrete at Capital:							
Diagonal Direction.	Maximum..	530	265	795	800	265	1 065
	Average....	500	250	750	750	250	1 000
Cross Direction.	Maximum..	500	250	750	800	250	1 050
	Average....	468	234	700	750	234	984

Concrete Stresses based on E_c = 1,875,000 lb. per sq. in.

In making up this table the dead load stresses have been taken as one-half the indicated live load stress at the design load (the dead weight of the slab being half this load). This is a maximum assumption and probably is somewhat in excess of the true value, as the concrete was not broken in tension until after a live load of 75 lb. per sq. ft. was applied.

55. *Cracks.*—Very careful observations were made to discover and record all cracks. A reading glass was used to aid the eye, and dust was removed by means of bellows. It is easy *not* to discover cracks in such a test, and with casual observations very likely but few of these cracks would be noted. In fact all of them were very fine. At a load of 262.5 lb. per sq. ft. a crack was observed at the bulkhead where two days had elapsed between the pouring of the adjacent floor sections.

At 300 lb. per sq. ft. cracks appeared at the other bulkheads. Very fine cracks were also found in the center panel where no bulkhead existed and over the edge of the capital at column No. 41, these being very faint and hard to trace for any distance. At 350 lb. per sq. ft. there could be traced out the cracks shown in Fig. 75 in which the dotted lines represent cracks in the ceiling below. The cracks about the column head are of interest as indicating the position of the critical

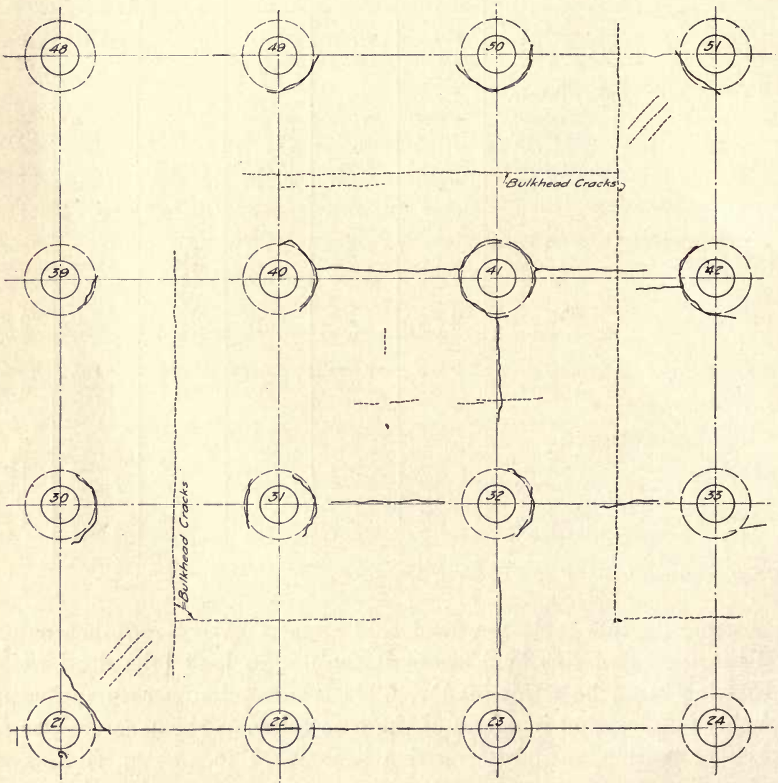

FIG 75. LOCATION OF CRACKS TRACEABLE AT LOAD OF 350 LB. PER SQ. FT.

section for which moments should be figured in analyses. These averaged about 2 or 3 in. outside the edge of the capital.

At column No. 51 the position of the crack would seem to indicate that for a single panel loaded the critical section moves nearer the support, resulting in higher stresses at the center. This crack and similar ones, as at columns No. 49, 39 and 21, were very faint, indicating a lower stress in the reinforcement over the support at such points. The cracks shown running diagonally near columns No. 21 and 51 were in all cases directly beneath slab rods.

Another set of cracks which developed only under the maximum load of 350 lb. per sq. ft. is significant. These cracks ran along the center line of the cross bands, being easily traced in the portion about half way between columns, growing fainter toward the columns, and disappearing entirely in most cases before reaching the crack over the edge of the capital. Evidently there is negative bending moment at these sections. These cracks, we believe, had not been observed before, probably because other building tests have not been so extensive, and because cracks have not ordinarily been very carefully observed.

56. *Comments.*—The most important result of the Deere and Webber Building test lay in the demonstration that a field test of a reinforced concrete building may be made with the reasonable expectation of securing reliable and useful data on the stresses developed in the steel and in the concrete. The test gives certain well-defined indications. It shows that the bending moment at the support is much greater than that at the center of the span. It indicates, by the position of the cracks, a critical section for which moments should be calculated. It indicates that the stresses at the center of the span are lower than analyses would lead one to expect. It indicates that bulkheads act to increase deflections and stresses. It indicates that the reinforcement at the center receives its maximum stress for the condition of load on one panel only.

VI. General Comments.

57. *General Comments.*—The tests described in the bulletin are of such a nature and cover so much ground that it is impracticable to summarize results or to formulate specific conclusions in any brief way. In the body of the text, the results of tests have been stated and described in detail, the action of structures discussed and conclusions drawn. The data are given in full in the tables and diagrams. In general the conclusions may be considered to be applicable to structures of similar construction. Possibly some of the conclusions, easily recognized in the text, will require further tests to determine whether they are generally appli-

cable. The information obtained in these tests will be found of value in the settlement of a number of questions which are in dispute, and the results when taken in connection with other tests may be expected to be of considerable assistance in developing analyses and determining constants for use in the design of reinforced concrete structures. Many of the results of the tests have a bearing upon the unsettled problems and even on matters which many have considered to be not open to question.

The tests here recorded have shown the practicability of measuring the deformations or strains in critical parts or members of a reinforced concrete structure when subjected to load. Methods have been developed for making measurements and tests in a way that will give trustworthy data. Difficulties have been overcome, and many of the precautions found necessary have been formulated. Skill and experience are essential in making such tests, and the difficulties encountered are of a wider range than those met in the best laboratory practice. As in other tests, caution must be exercised in drawing conclusions, and judgment must be used in interpreting results. The presence of low stresses should not be taken as being conclusively indicative of low bending moments; and the action of tension in the concrete, of horizontal thrust distributed over large distances, and of other agencies may need consideration. Action under partial load, as when a single panel is loaded, must be recognized to differ from that under full load.

Such problems as the distribution of bending moments along the length of the beam, the distribution of stresses over areas outside of those usually assumed as forming the beam, the presence of secondary stresses and of web stresses in structures as they are fabricated, will be solved only when adequate field tests have been made. The analysis of structures and the determination of the resistance of individual members or parts require the making of assumptions and the choice of constants, and the proper determination of these may be made only with full knowledge of the properties of the materials found by laboratory tests and of the action of the fabricated structure as shown in adequate field tests.

PUBLICATIONS OF THE ENGINEERING EXPERIMENT STATION

Bulletin No. 1. Tests of Reinforced Concrete Beams, by Arthur N. Talbot. 1904. *None available.*

Circular No. 1. High-Speed Tool Steels, by L. P. Breckenridge. 1905. *None available.*

Bulletin No. 2. Tests of High-Speed Tool Steels on Cast Iron, by L. P. Breckenridge and Henry B. Dirks. 1905. *None available.*

Circular No. 2. Drainage of Earth Roads, by Ira O. Baker. 1906. *None available.*

Circular No. 3. Fuel Tests with Illinois Coal (Compiled from tests made by the Technologic Branch of the U. S. G. S., at the St. Louis, Mo., Fuel Testing Plant, 1904–1907), by L P. Breckenridge and Paul Diserens. 1909. *Thirty cents.*

Bulletin No. 3. The Engineering Experiment Station of the University of Illinois, by L. P. Breckenridge. 1906. *None available.*

Bulletin No. 4. Tests of Reinforced Concrete Beams, Series of 1905, by Arthur N. Talbot. 1906. *Forty-five cents.*

Bulletin No. 5. Resistance of Tubes to Collapse, by Albert P. Carman. 1906. *Fifteen cents.*

Bulletin No. 6. Holding Power of Railroad Spikes, by Roy I. Webber. 1906. *Thirty-five. cents*

Bulletin No. 7. Fuel Tests with Illinois Coals, by L. P. Breckenridge, S. W. Parr, and Henry B. Dirks. 1906. *Thirty-five cents.*

Bulletin No. 8. Tests of Concrete: I. Shear; II. Bond, by Arthur N. Talbot. 1906. *None available.*

Bulletin No. 9. An Extension of the Dewey Decimal System of Classification Applied to the Engineering Industries, by L. P. Breckenridge and G. A. Goodenough. 1906. Revised Edition 1912. *Fifty cents.*

Bulletin No. 10. Tests of Concrete and Reinforced Concrete Columns, Series of 1906, by Arthur N. Talbot. 1907. *None available.*

Bulletin No. 11. The Effect of Scale on the Transmission of Heat through Locomotive Boiler Tubes, by Edward C. Schmidt and John M. Snodgrass. 1907. *None available.*

Bulletin No. 12. Tests of Reinforced Concrete T-beams, Series of 1906, by Arthur N. Talbot. 1907. *None available.*

Bulletin No. 13. An Extension of the Dewey Decimal System of Classification Applied to Architecture and Building, by N. Clifford Ricker. 1907. *Fifty cents.*

Bulletin No. 14. Tests of Reinforced Concrete Beams, Series of 1906, by Arthur N. Talbot. 1907. *None available.*

Bulletin No. 15. How to Burn Illinois Coal without Smoke, by L. P. Breckenridge. 1908. *Twenty-five cents.*

Bulletin No. 16. A Study of Roof Trusses, by N. Clifford Ricker. 1908. *Fifteen cents.*

Bulletin No. 17. The Weathering of Coal, by S. W. Parr, N. D. Hamilton, and W. F. Wheeler. 1908. *Twenty cents.*

Bulletin No. 18. The Strength of Chain Links, by G. A. Goodenough and L. E. Moore. 1908. *Forty cents.*

Bulletin No. 19. Comparative Tests of Carbon, Metallized Carbon, and Tantalum Filamen Lamps, by T. H. Amrine. 1908. *Twenty-five cents.*

Bulletin No. 20. Tests of Concrete and Reinforced Concrete Columns, Series of 1907, by Arthur N. Talbot. 1908. *None available.*

Bulletin No. 21. Tests of a Liquid Air Plant, by C. S. Hudson and C. M. Garland. 1908 *Fifteen cents.*

Bulletin No. 22. Tests of Cast-Iron and Reinforced Concrete Culvert Pipe, by Arthur N Talbot. 1908. *Thirty-five cents.*

Bulletin No. 23. Voids, Settlement, and Weight of Crushed Stone, by Ira O. Baker. 1908. *Fifteen cents.*

Bulletin No. 24. The Modification of Illinois Coal by Low Temperature Distillation, by S. W Parr and C. K. Francis. 1908. *Free upon request.*

Bulletin No. 25. Lighting Country Homes by Private Electric Plants, by T. H. Amrine. 1908 *Free upon request.*

Bulletin No. 26. High Steam-Pressures in Locomotive Service. A Review of a Report to the Carnegie Institution of Washington, by W. F. M. Goss. 1908. *Free upon request.*

Bulletin No. 27. Tests of Brick Columns and Terra Cotta Block Columns, by Arthur N. Talbo and Duff A. Abrams. 1909. *Free upon request.*

Bulletin No. 28. A Test of Three Large Reinforced Concrete Beams, by Arthur N. Talbo 1909. *Free upon request.*

Bulletin No. 29. Tests of Reinforced Concrete Beams: Resistance to Web Stresses, Series of 1907 and 1908, by Arthur N. Talbot. 1909. *Free upon request.*

Bulletin No. 30. On the Rate of Formation of Carbon Monoxide in Gas Producers, by J. K Clement, L. H. Adams, and C. N. Haskins. 1909. *Free upon request.*

PUBLICATIONS OF THE ENGINEERING EXPERIMENT STATION

Bulletin No. 31. Fuel Tests with House-Heating Boilers, by J. M. Snodgrass. 1909. *Free upon request.*

Bulletin No. 32. The Occluded Gases in Coal, by S. W. Parr and Perry Barker. 1909. *Fifteen cents.*

Bulletin No. 33. Tests of Tungsten Lamps, by T. H. Amrine and A. Guell. 1909. *Free upon request.*

Bulletin No. 34. Tests of Two Types of Tile Roof Furnaces under a Water-Tube Boiler, by J M. Snodgrass. 1909. *Free upon request.*

Bulletin No. 35. A Study of Base and Bearing Plates for Columns and Beams, by N. Clifford Ricker. 1909. *Twenty cents.*

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