

Observed Deflections of Reinforced Concrete Slab Systems, and Causes of Large Deflections

By ACI Committee 435

D. R. Buettner*
Chairman

S. K. Ghosh*
Chairman, Subcommittee on Field
Measurements

D. E. Branson
R. G. Drysdale*
A. Farah
A. B. Gogate
J. Grossman
C-T. T. Hsu

S. V. Kulkarni
M. S. Mirza*
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M. V. Pregnoff
G. M. Sabnis*
C. G. Salmon

A. Scanlon*
J. M. Spang
M. K. Tadros
A. F. Shaikh
S. Zundeleovich

***Members of the Subcommittee on Field Measurements which prepared this report.**

****Please see Preface for the entire 435 membership.**

Synopsis: This report is in two distinct parts

Part I is a summary of published studies on slab deflections (3 from Australia, 1 from Scotland, 1 from Sweden, 2 from U.S.). The summary focuses on construction practices and materials quality. Comparison of deflections calculated by various methods with actual long-term deflections is made in some cases.

Part II summarizes several construction problems and material deficiencies which can contribute to large long-term deflections. Focusing on large construction loads, the authors show

that construction loads may be considerably higher than design loads and that high construction loads cause high initial deflections because concrete has a lower modulus of elasticity when loaded at an early age. Furthermore, concrete creeps more when it is loaded at an early age, thereby causing additional high long-term deflections, even when construction loads are sustained only for a few days.

The authors then suggest a method of form removal and reshoring that has proved successful in the New York City area in preventing large slab deflections. Essentially, no more than an 8-foot slab span is left unsupported until a slab is mature.

A reader interested only in the Committee's findings and recommendations may proceed straight to Part II of the report.

Keywords: concrete construction; concrete slabs; creep properties; deflection; flat concrete plates; form removal; loads (forces); modulus of elasticity; reinforced concrete; shoring; shrinkage; two-way slabs.

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Part I

FIELD DEFLECTION MEASUREMENTS OF REINFORCED CONCRETE
FLAT PLATES, FLAT SLABS AND BEAMS: A REVIEW OF LITERATURE

This part of the report reviews and summarizes the existing literature on field deflection measurements of reinforced concrete flat plates, flat slabs and beams.

INVESTIGATION A

Summary

Three experimental flat plate structures were erected at the Division of Building Research, Commonwealth Scientific and Industrial Research Organization, Melbourne, Australia. The investigations were carried out under field conditions, the structures being completely exposed to the weather.

Structure Mark I consisted of an expanded shale concrete slab, 3-1/2 in. thick, spanning three bays of 9 ft in one direction and three bays of 12 ft in the other, with cantilevers 4 ft 6 in. long in this direction. The reinforcement was conventional individual plain round bars and was designed by the empirical design method given in ACI 318-56. The slab was carried on 16 steel columns of box section with a grillage type shear connection. The significant features of this structure were (1) span/depth ratios of 41 in one direction and 31 in the other; (2) the ratio 4:3 of the sides of the panels; and (3) the steel columns. The use of lightweight aggregate concrete was also an important feature. Further, no edge beams or torsion reinforcement near the edge columns was used.

The long-term deflections reached "annoying" proportions. The slab was allowed to stand under its own weight for 8 months, during which time the deflection at the center of the middle panel increased by 0.62 in. This was 12 times the initial elastic deflection of 0.05 in. In a study of the long-term deformation of this structure it was suggested that about 20% of the increase at the center of the middle panel was due to differential settlement of inner and outer columns, about 40% was due to further cracking causing a reduction in stiffness, and to local bond slip, and about 40% to creep. This analysis also suggested that the increment of deflection due to creep was about 85% of the immediate deflection of a completely cracked slab..

In connection with the large long-term deformations, three features were pointed out. First, the structure was constructed of expanded shale concrete. Available evidence suggests that in

concrete made with well-coated, expanded shale aggregate, the creep may in certain cases be 20% greater than for natural rock concrete at the same stress, which would be an insignificant contribution in this case. Secondly, the experimental structure was built in the summer, and during its early history was exposed regularly to high ambient temperatures and direct sunlight. It has been shown that creep is directly proportional to temperature, for set cement pastes. Finally, since the structure was outdoors, completely exposed, it was under widely fluctuating conditions of temperature and relative humidity. Creep and shrinkage under fluctuating conditions have been shown to be greater than under constant average conditions of storage.

Structure Mark II had 9-ft spans over two bays in one direction and three bays in the other. Cantilevers 3 ft long extended in the two-span direction. The slab was of expanded shale concrete and was intended to be 4-in. thick, but because of distortion of the formwork it was much thicker in some places. The concrete, supplied by an outside contractor, contained in error some dense basalt in addition to the expanded shale aggregate. These two factors combined to make the slab much stiffer than was intended and useless for studies of deformation. No attempt, therefore, was made to examine its deflection under imposed loading, and it was tested directly to destruction.

Structure Mark III, probably the first prestressed, post-tensioned flat plate in Australia, was allowed to stand under its own weight to obtain data on loss of prestress.

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- A.1a. "Experimental Flat Plate Structure of Expanded Shale Concrete," *Constructional Review* (Sydney), Vol. 33, No. 2, Feb. 1960, pp. 22-29.
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- A.1c. "Experimental Lightweight Flat Plate Structure, Part II - Deformations due to Self Weight," *Constructional Review* (Sydney), Vol. 34, No. 3, Mar. 1961, pp. 25-33.
- A.1d. "Experimental Lightweight Flat Plate Structure, Part III - Long-Term Deformations," *Constructional Review* (Sydney), Vol. 34, No. 4, Apr. 1961, pp. 21-26.

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INVESTIGATION B

Summary

The paper (**Ref. B.1**) describes large deflections found in electrically heated reinforced concrete floor slabs in a considerable number of Scottish apartment buildings and gives the basic reasons why such deflections occurred. The slabs were supported on three sides on load-bearing walls, and were free along the fourth edge. The slabs had noticeable deflections along their free edges. Some of the deflections were up to 1.25 in. in a clear span of 12 ft 5-1/2 in.

The paper describes a number of laboratory tests to ascertain the shrinkage of the aggregates used for constructing the floors and the influence on concrete made with such aggregates and similar sands. The results of modulus of elasticity tests are given. An investigation into the temperature and deflection

characteristics with or without live load of a typical apartment floor is described and the results are given, together with examples of deflection readings from other apartment floors and core crushing results.

Full-scale laboratory tests were set up using pairs of slabs cast with shrinkable Scottish and unshrinkable flint aggregate concretes operating at a range of temperatures.

The conclusions drawn were as follows:

- (1) **Shrinkage of Aggregates** - Almost all the aggregates tested from the lowlands of Scotland gave rise to higher concrete shrinkage than are expected from good aggregate, e.g. flint. Depending on the degree of this shrinkage, the deflection of members constructed with such aggregates will be greater than when unshrinkable ones are used.
- (2) **Effect of Floor Heating** - Under-floor heating caused partial drying out of the slabs and was responsible for a small part of the deflections that had occurred.
- (3) **Modulus of Elasticity** - It is essential that designers check the deflection likely to occur in a beam or slab by considering the instantaneous E and the anticipated shrinkage and creep values of the concrete being used, and assessing a value for the effective E which can be used in the normal formulae. The long-term deflection of the test apartment floor was approximately 12 times the estimated instantaneous deflection under dead and live load. In the full-scale laboratory tests the movement at the time of writing was up to approximately 8 times the instantaneous values.
- (4) **Use of Top Reinforcement** - The full-scale tests showed that a considerable reduction can be made in the deflections of slabs by use of a suitable quantity of top reinforcement. In continuous slabs a large amount of top steel is used for continuity and this is, no doubt, partly the reason such slabs suffer fewer problems. Cantilevers are usually unsatisfactory.
- (5) **Span/Depth Ratios** - The results obtained point very strongly to the need for a close reexamination of the recommended values of span/depth ratios given in many codes of practice. Under conditions where the full design live load is likely to be supported for long periods, especially when combined with simply supported spans, a significant reduction in such permitted ratios may be necessary.

Reference

- B.1 Jenkins, R.A.S., Plowman, J.M. and Haseltine, B.A., "Investigation into the Causes of the Deflection of Heated Concrete Floors, Including Shrinkage," The Structural Engineer (London), Vol. 43, No. 4., April 1965, pp. 105-117.

INVESTIGATION C

Summary

The report (**Ref. C.1**) summarizes the results of a field investigation to determine the short- and long-time deflections and concrete strains in an Army barracks flat-plate structure at Ford Hood, Killeen, Texas.

Due to the rather great slab thickness of 9 in., corresponding to a span-to-depth ratio of approximately 28, all observed deflections were small and in no instance exceeded 0.022 ft, or about 1/800 of the shorter span, during the 45-month observation period, in spite of an early temporary construction load estimated to have been almost 30 percent in excess of the total design load.

The measured short-time deflections under various loading conditions compared reasonably well with deflections predicted by use of the Ersatz frame analysis method (as proposed by Vanderbilt, Sozen and Siess).

Reference

- C.1 Geymayer, H.G., and McDonald, J.E., "Short-and Long-time Deflections of Reinforced Concrete Flat Slabs," Technical Report C-70-1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, February 1970, 9 pp. plus 4 tables with 17 figs.
- C.2 Vanderbilt, M.D., Soren, M.A., and Siess, C.P., "Deflections of Reinforced Concrete Floor Slabs," Structural Research Series No. 263, Civil Engineering Department, University of Illinois, Urbana, Illinois, April 1963.



INVESTIGATION D

Summary

Field deflection measurements were taken on five buildings in and around Sydney, Australia for periods of up to 9 years. The buildings and some of the results of these investigations are described below. Some recurring design and construction problems, pointed up by these investigations, are enumerated.

Structure I (Refs. **D.1**, **D.2**, **D.3**, Building No. 1 of Ref. **D.8** "Taylor's Flat Plate" of Ref. **D-9**, Structure 1 of Ref. **D.10** Building 1 of Ref. **D.11**) - The dimensions are shown in **Fig. 1a**. The typical interior panel of 20 ft 10 in. x 16 ft 8 in. with the 8 in. thick plate gives a longer span to depth ratio of 31. The slab was designed for a dead load of 110 psf and superimposed load of 75 psf. The concrete design strength was $f'_c = 3$ ksi. Typical reinforcement was of hard grade deformed bars (f_y min = 50 ksi); each mid-panel had a layer of top reinforcement of welded wire fabric (f_y subscript = 70 ksi). The deflection behavior of the slab was expressed by the equation:

$$\text{Long term deflection} = \mathbf{A+B+C+D+E+F}$$

- where A = initial elastic deflection (13% of total) caused by slab dead load on removal of props.
- B = long-term elastic deflection (1-1/2%) caused by superimposed loads and finishes, without producing areas of cracked section in the slab.
- C = initial cracking deflection (1/2%) due to production of cracked sections in the concrete slab at the time of prop removal.
- D = long-term cracking deflection (19%) due to transformation of slab from uncracked to partially cracked as construction loads occurred or as tensile failure was initiated in the concrete under sustained load and drying shrinkage.
- E = shrinkage deflection (26%) caused because the shrinkage restraint afforded by the different quantities of reinforcement at the top and bottom surfaces of the slab was unequal.
- F = creep deflection (40%).

The long-term deflection curves are shown in Figs. **1b**, **c**, **d**, and **e**. The ratio of long-term to initial deflection for structure I was 6.7, significantly greater than the generally accepted value of 3. The initial deflection of Structure I comprised only the elastic deflection under slab dead load, whereas the multiplier of 3 was derived from five different investigations on 68 model beams and one-way slabs with extreme fiber concrete compressive stresses of approximately 0.45 f'_c at initial deflection.

Hence the initial deflections of the models comprised the total elastic deflection plus the deflection due to short-term cracking, and therefore formed a larger percentage of the final deflection, resulting in a lower value for the ratio of long-term to initial deflection than that obtained for the test structure.

The models mentioned above were made from concrete with American mix constituents. The concrete probably had lower shrinkage and creep characteristics than the commercially available Sydney ready-mixed concrete used in Structure I. A series of Sydney laboratory shrinkage tests made on 3x3x11 in. prisms at 50% relative humidity for 1 year gave shrinkage measurements of 670 to 930 micro in./in. The corresponding values for typical European concrete are 210 to 500 micro in./in. Hence shrinkage deflection, which comprised 26% of the total long-term deflection calculated by the method proposed in Ref. D.3, was probably higher than normal, and this may have partially contributed to the high multiplier value obtained.

The measured long-term deflection for Structure I was 3 to 5 times the values calculated by various available methods. Such large discrepancies between the deflection that is expected to occur in a building and the deflection that actually does occur would normally create serious problems with partitions in an apartment or office building. The measured long-term deflection for Structure I was 1.2 times the value calculated by the Taylor method (Ref. D.3). This degree of accuracy would be sufficient to enable designers to avoid long-term deflection problems.

Structure II (Ref. D.4, Building No. 2 of Ref. D.8, "Taylor's Flat Slab" of Ref. D.9, Structure 2 of Ref. D.10, Building 2 of Ref. D.11) - The area investigated formed part of Level 2 of a large reinforced concrete flat slab in an unenclosed car park. The car park was on three levels - Level 1 on the ground and Levels 2 and 3 suspended. The slab thickness was 9-1/2 in. with 8 ft x 8 ft x 6 in. drop panels; a typical bay size was 28 ft 3 in. x 26 ft. The columns were 20 in. x 20 in. The test area formed part of a section 133 ft x 158 ft that was separated from the adjacent car park area by contraction joints (Fig. 2a). The longer span-to-depth ratio was 36. The design load used was 120 psf dead load and 60 psf car parking live load. The concrete design strength was $f'_c = 3$ ksi. The reinforcement used was hard-grade deformed bars with $f_y \text{ min} = 50$ ksi. The average compressive strength of thirty 6 in. cylinders tested at 28 days was determined to be 5600 psi.

The long-term deflection curves are shown in Figs. 2b and c. The ratio of long-term to initial deflection was 8.7 for points 2, 7. The comments for Structure I apply to this structure also. Shrinkage aggravated by poor curing and windy conditions at the time of placement, was thought to be a major factor contributing to the large long-term deflections.

Structure III (Ref. **D.6**, Building No. 4 of Ref. **D.8**, "Heiman's flat Plate" of Ref. **D.9**, Structure 3 of Ref. **D.10**, Building 4 of Ref. **D.11**) - The flat plate forms Level 1 of an unenclosed car park in a motel building. There are two floors of beam-and-slab construction above the test slab. The dimensions are shown in Figs. **3a** and **b**. The slab is 9-1/2 in. thick with a typical interior panel size of 24 ft 9 in. x 23 ft 9 in. The longer span-to-depth ratio for an interior panel is 31. The design loading was: slab dead load, 115 psf and car park live load, 60 psf. The design concrete strength was $f'_c = 3$ ksi. Cold worked deformed bars were used for the reinforcement ($f_y \text{ min} = 60$ ksi). The long-term deflections are shown in Figs. **3c**, **d** and **e**. The figures show that slab deformation commenced from the time of casting and, by the time the props were removed, was a significant portion of the 600-day deflection. Although the props appeared to have adequate support on sole plates, wet weather reduced the bearing capacity of the ground at about the time of casting, permitting settlement to occur. Shrinkage was again thought to have played an important role. A significant part of the long-term deflection was thought to have been caused by a loss of stiffness resulting from cracking, brought about by a combination of flexural and shrinkage stresses together with the additional stresses induced by prop settlement.

Structure IV (Ref. **D.7**, Building No. 3 of Ref. **D.8**) - The structure is a circular building of 50 stories and is 600 ft high. It is 135 ft in diameter and has a circular service core 62 ft in diameter. The structural system used in most of the floors consists of 20 radial, tapered beams spanning 35 ft between the core and the 20 external columns which project beyond the window line. The beams are spaced at approximately 15 ft centers at midspan, and have an overall depth of 20-1/2 in. The slab thickness is 4-5/8 in. Normal weight concrete was used only in the footings and in the columns and walls up to the eighth level. Lightweight concrete was used elsewhere.

The time of withdrawal of the props under the beams ranged from 25 to 32 days after placement of the concrete. A polyvinyl acetate based curing compound was sprayed on the slabs after placement of the concrete. The average compressive strength of the concrete taken from the four test floors, where measurements were carried out, was 3560 psi at 7 days and 4670 psi at 28 days. However, the average compressive strength of five drilled cores taken from the in situ concrete at about 9 months and tested dry was only 4000 psi. The average modulus of elasticity of these cores was 2.3×10^6 psi. The average shrinkage of 3 x 3 x 11 in. concrete prisms was 600 micro in./in. at 90 days.

An investigation of the magnitude of construction loads transmitted through propped floors and the variations in this loading during the construction cycle was carried out. Four load cells were placed under some of the props on the 38th floor in the vicinity of the intersection of two secondary ring beams with

one of the radial beams. The measurements showed that the construction loading increased very rapidly after the props were removed from under the test floor and reached a peak of 121 psf when it was supporting three floors above together with the associated formwork.

The 38th floor was placed in mid-summer during a period of high temperatures, and the rate of strength development of the concrete was probably greater than during the winter months when the 16th and 20th floors were placed. A comparison of the deflections for the three floors during the construction period indicated that the amount of load passed by two propped floors to a lower test floor prior to removal of the props was higher during the winter than was shown by the load cell measurements. Construction loads well in excess of the design value of 100 psf were probably applied to the beams on the lower floors after the props below were removed.

A test load of 100 psf, corresponding to the design live load, was applied over two bays of the 24th floor. Bricks were stacked on the slab 3-1/2 months after the concrete was placed and the load was left on the slab for 6 months. The graph showing the measured deflection plotted against time, along with other deflection measurements, is shown in Fig. 4. It can be seen that the deflection that occurred when the test load was applied (0.13 in.) was much less than the maximum deflection that occurred during the construction period (0.33 in.) Also, at 280 days when the test load was removed, the recovery was about the same as the elastic deflection that took place at the time of application of this test load, whereas the measured recovery at the time of removal of the construction loading was only 0.01 in. The difference in these recoveries could be accounted for by the fact that the stress/strength ratio was higher at the time of application of the construction loading than when the test load was applied, and more cracking occurred during the initial application of load than during the application of test load.

The long-term deflections of a flexural member are influenced significantly by the degree of cracking that takes place and the associated loss of stiffness. Magnitude of construction loads, variations in ambient temperature and humidity during and soon after the placement of the concrete, rate of strength development of concrete, time of removal of props, and method of curing do all have some effect on the amount of cracking that takes place during the early history of the members, and on the amount of recovery that occurs when the construction loads are removed. None of these factors is taken into consideration in the usual methods of calculating long-term deflection; consequently, there are often marked differences between calculated and measured deflections.

Calculated deflections were generally found to be less than the measured deflections. The difference between the measured



deflection and calculated deflection (ACI Code) represented an error of 50%. The differences have been much greater in flat plate and flat slab structures.

Although large deflections of the beams were measured during the construction period, the total deflection-to-span ratios were small. The disadvantages associated with the increased shrinkage and lower elastic modulus of lightweight concrete were successfully offset by the use of a large amount of top steel and an adequate span-to-depth ratio. Furthermore, the higher modular ratio associated with lightweight concrete results in increased stiffness, a lower neutral axis, and consequently lower concrete stresses and creep compared with normal weight concrete.

Structure V (Ref. D.5, structure 4 of Ref. D.10) - This structure was measured for some 6 months before it was realized that deflection had ceased, as shown in Fig. 5. It was discovered that the floor and roof slabs in each of the six stories of the building had deflected onto the brick partitions of the floors below, because insufficient gaps had been left above the partitions to allow for long-term deflection. Although the real long-term deflection could not be measured, the investigation revealed a serious construction problem, which resulted in the lower story brickwork being overstressed.

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INVESTIGATION E

Summary

Deflections of concrete floor slabs were studied and measured in two apartment houses which were constructed in similar ways (Figs. 6a and c). The structural framework was made of cast-in-situ reinforced concrete and comprised floor slabs spanning in one direction and transverse walls supporting the floors. The spans were 20 ft 2 in. and 22 ft 4 in. respectively, and the total slab depths 8 in. and 9 in. In both cases large formwork units were used: vertical free standing wall forms and horizontal table forms. The table forms were removed 4-8 days after casting. In order to reduce the deflections, props were mounted between the floors, and these props were not removed until 18-28 days after casting. Measurements were taken by means of a levelling instrument on a total of 20 slabs. Each slab was first levelled before the formwork was struck, and the second levelling

was carried out soon after; Later on, levels were taken at increasing intervals. In test series 1, span 20 ft 2 in., the measurements continued over about three months, while the measurements in test series 2, span 22 ft 4 in., covered a period of about two years. In order to check the concrete quality the compressive strength was measured on 6 in. standard cubes, and by rebound hammer on the floors. Shrinkage and creep deformations were measured on special prisms. All the test specimens were stored at the building site to have the same climatic conditions as the slabs.

The deflections measured in test series 1 are plotted in Fig. 6b which shows the measurements on all the slabs and a calculated average curve. Fig. 6d is similar to Fig. 6b, and shows the results of measurements from test series 2.

In both figures (6b and 6d), there is a considerable scatter around the arithmetic average of the measured deflections. The size and time dependence of the deflection are affected by several individually varying factors: for instance, different concrete ages at removal of formwork, different standards of workmanship and different loading conditions. It was observed that the deflections were considerably less for the top floor slabs. This difference was mainly due to different loading conditions, since the top floors did not support any floors above during the erection of the buildings. Since the span and the span-to-thickness ratio were approximately the same for the two buildings studied, a comparison might be of interest. A direct comparison of the average deflections from the two test series is made in Fig. 6e and it can be seen that the deflection curves only deviate slightly from each other.

The final deflection for the floors on which measurements were taken in series 2, including the top floor, was estimated at 0.53 in. or $1/500$ of the span. About one-third of the time-dependent deflection had taken place after two months when the partitions were erected in the actual building. The measurements showed that an average deflection of 0.20 in. had occurred at this time, and therefore the remaining deflection after the erection of partitions amounted to only 0.33 in. or $1/800$ of the span.

Several calculation methods for deflections were studied. The methods were applied to the floor slabs studied in series 2. The results showed final deflections varying over a wide range, between 0.20 in. and 1.77 in.