

State-of-the-Art Report on Temperature-Induced Deflections of Reinforced Concrete Members

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Synopsis: This report summarizes available methods for calculating deflections of reinforced concrete beams subjected to temperature change. Selection of design temperatures and temperature gradients is discussed as well as the effects of cracking on response in the service load range.

Keywords: beams (supports); cracking (fracturing); deflection; modulus of elasticity; moments of inertia; reinforced concrete; temperature; thermal gradient.

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INTRODUCTION

Temperature changes can significantly affect deflections of reinforced concrete building structures. Deflections occur in unrestrained flexural members when a temperature gradient is set up between opposite faces of the member. In cases where deformations due to temperature change are restrained, tensile stresses induced in the member can result in cracking and consequent reduction in flexural stiffness. Because temperature effects do not often affect the ultimate limit state of the structure, effects of temperature on deflection are sometimes not considered in design. However, it has been standard practice to compute thermal stresses and displacements in structures for tall building design. Also, elongation and shortening of bridge superstructures and precast concrete structures are normally computed for support and expansion joint designs.

De Serio (1) states that "Designing for thermal and shrinkage stresses is the most neglected part of today's design practice." With the use of higher strength materials and more refined methods of analysis the need to consider temperature effects is becoming increasingly important.

Most general purpose computer programs have the capability to include temperature changes in the analysis for certain types of members. However, in many cases, particularly those involving relatively simple structures, where computer analysis is not required, or where the computer program does not include the capability for analysis of flexural members subjected to temperature gradient, calculations may be required to investigate the effect of temperature change on serviceability.

The objective of this report is to indicate some of the problems that can result from differential thermal movement and to outline procedures for calculating deflections that result from temperature change. The scope of the report is restricted to performance of structures in service. Temperature effects due to heat of hydration are not considered.

SERVICEABILITY PROBLEMS RELATED TO DIFFERENTIAL THERMAL MOVEMENT

The following examples illustrate potential problems that can result from differential thermal movement in building structures.

Office building - cantilever roof and floor slabs. Daily temperature changes caused deflections of about one inch at the cantilever ends of roof slabs in a four-story building constructed of cast-in-place concrete. The post-tensioned floors and roof were 80 feet by 80 feet in plan. Only four columns were provided and these were 48 feet apart in each direction which resulted in the columns being 16 feet from the edges of the floor and roof slabs in each direction. During construction it was noted that diurnal solar heating caused the roof slab to deflect up and down. The change in elevation was about one inch at the extreme overhanging

ends. Fortunately the detail for the connection of the windows, which were located in the perimeter of the floors and roof, provided for sufficient movement that the windows remain attached to the roof when it is in its highest position and does not result in the window becoming loaded when the roof is in its lowest position.

Industrial building - cantilever roof slab. Similar temperature-induced movements caused damage at the joint between a cantilever roof slab and exterior wall in a one-storey building that was erected using the lift-slab technique. The post-tensioned waffle roof slabs had significant overhangs at all four exterior walls. The exterior walls were constructed using the tilt-up method. Because of the construction techniques used, the roof was not supported vertically by the exterior walls and the connection of the exterior walls (for horizontal loads) was made when the edges of the roof were in a low position due to the effects of diurnal solar heating. During service the effect of solar heating has been to apply vertical loads to the exterior walls. This has resulted in the vertical cast-in-place concrete closure pours between the wall panels, particularly at the corners of the building, to crack and disintegrate near their bottoms with the passage of time. Reconstructing the destroyed concrete does not solve the problem; it simply disintegrates again. Eventually, attempts to reconstruct the disintegrated concrete were abandoned and steel plates, painted to match the color of concrete, were placed on the affected joints to cover the damage and improve the appearance of the building.

Parking structure - deflection of double tee beams. In a precast parking structure, rotation at bearing ends of beams resulting from deflections caused by diurnal solar heating produced cracking near the ends of the beams. The precast prestressed double-tee beams span about 55 feet. A concrete topping was placed over the top of the double-tee beams. Reinforcing steel was placed in the topping at interior joints in a configuration similar to that used with negative moment reinforcement in cast-in-place construction. At some locations elastomeric bearing pads were placed between the stems of the double-tee beams and their supports; at others, elastomeric bearing pads were not provided under the stems. Measurements showed the deflection caused by diurnal solar heating to be of the order of 0.75 inches at midspan of the double-tee beams. The deflection due to solar heating caused rotations at the ends of the double-tee beams and many of the double-tee beam stems, not provided with elastomeric bearing pads, cracked as a result of the rotation.

Office building - vertical wall panels. Exterior precast wall panels two stories high were supported at the bottom and supported laterally at the top of the second storey. In some locations the panels were placed adjacent to wall construction supported laterally at the top and bottom of the second storey. Seasonal temperature changes caused bowing of the two storey panels resulting in mid panel deflections up to 3/4 in. Relative deflection between the two-storey panels and adjacent walls supported at the bottom of the second storey caused damage to the

caulking material in the vertical joint.

These examples illustrate the need to consider differential thermal movements in design. The following sections outline calculation procedures that can be used to estimate stiffness changes and deflections resulting from temperature change in structural members.

DESIGN TEMPERATURES

Before performing an analysis for temperature effects it is necessary to select design temperatures and temperature gradients for use in the analysis. Martin (2) summarizes design temperatures that are provided in various national and foreign codes. Design information is given in terms of temperature rise ranging from 27 to 40°F and temperature drop ranging from 27 to 45°F. Martin also provides extreme values of normal daily maximum and minimum temperatures as reported by the Environmental Data Service for various locations in the United States. He suggests that design should be based on 2/3 of the difference between extreme values of normal daily maximum and minimum temperatures for each location. Based on this criterion average proposed design thermal differential is 41°F with a minimum of 10 and a maximum of 62°F. Fintel and Khan (3) suggest contacting the local weather bureau to obtain mean temperatures for specific localities. They also point out that surface temperatures can be affected by direct radiation. This may be significant, particularly for thin members. Fintel and Khan provide a method to determine isotherms and temperature gradients for irregular configurations and boundary conditions. They show that nonlinear thermal gradient can induce internal stresses in the member even if the member is free to deform at the ends.

A design thermal gradient used in New Zealand for concrete bridge superstructures is shown in Fig. 1. A fifth power temperature profile is used between the top surface and a point at depth 1200 mm for webs and cantilever decks. For decks above enclosed air-cells, a linear gradient is used. A linear thermal gradient is also used for the bottom 200 mm of a section. For superstructures less than 1400 mm in depth, the top and bottom thermal gradients are superimposed.

Charts showing temperature gradients in slabs of different thickness exposed to sunlight on a summer day in Melbourne, Australia are given in Ref. 4. These charts show approximately linear temperature gradients and a temperature difference of about 40°F over the depth of an 8-in. slab.

Fintel and Khan (3) provide a graphical method for determining temperature gradients for given steady state temperatures. Temperature gradients can also be determined by the finite element method (5), or the finite difference method (6).

TEMPERATURE GRADIENT ON UNRESTRAINED CROSS SECTION

Consider a temperature distribution $t(y)$ on the cross section shown in Fig. 2(a). Thermal strain at distance y from the bottom of the section is given by:

$$\epsilon_t(y) = \alpha t(y) \quad (1)$$

To restrain movement due to temperature $t(y)$, apply a stress in the opposite direction to $\epsilon_t(y)$:

$$\sigma(y) = E\alpha t(y) \quad (2)$$

The net restraining axial force and moment are obtained by integrating over the depth:

$$P = \int_A \sigma dA = \int_0^h \alpha E t(y) b(y) dy \quad (3)$$

$$M = \int_A \sigma(y-n) dA = \int_0^h E\alpha t(y) b(y) (y-n) dy \quad (4)$$

To obtain total strains on the unrestrained cross section, as shown in Fig. 2(d), apply P and M in the opposite direction to the restraining force and moment. Assuming plane sections remain plane, axial strain (ϵ_a) and curvature (ϕ) are given by:

$$\epsilon_a = \frac{P}{AE} = \frac{\alpha}{A} \int_0^h t(y) b(y) dy \quad (5)$$

$$\phi = \frac{M}{EI} = \frac{\alpha}{I} \int_0^h t(y) b(y) (y-n) dy \quad (6)$$

The net stress distribution on the cross section is given by:

$$\sigma_n(y) = \frac{P}{A} \pm \frac{M(y-n)}{I} - E\alpha t(y) \quad (7)$$

For a temperature gradient varying linearly from 0 to Δt , the curvature obtained by Eq. (6) is given by

$$\phi = \frac{\alpha \Delta t}{h} \quad (8)$$

In the case of a uniform temperature gradient along the length of a member, deflections for simply supported (Δ_{ss}) and cantilever beams (Δ_{cant}) are calculated as

$$\Delta_{ss} = \frac{\phi L^2}{8} = \frac{\alpha \Delta t}{h} \cdot \frac{L^2}{8} \quad (9)$$

$$\Delta_{cant} = \frac{\phi L^2}{2} = \frac{\alpha \Delta t}{h} \cdot \frac{L^2}{2} \quad (10)$$

Deflection to span ratio is given by

$$\frac{\Delta}{L} = \frac{\alpha \Delta t}{k} \cdot \frac{L}{h} \quad (11)$$

where $k = 8$ for simply supported and 2 for cantilever beams.

EFFECT OF RESTRAINT TO THERMAL MOVEMENT

If a member is restrained from deforming under the action of temperature changes, internal stresses will be developed. Cracking that occurs when tensile stresses exceed the concrete tensile strength reduces the flexural stiffness of the member and results in increased deflections under subsequent loading. Temperature effects should therefore be taken into account when determining member stiffness for deflection calculation.

Priestly (7) points out that an iterative solution is necessary to solve for the final strain distribution in cracked sections since the extent of cracking is a function of the thermal load. However for design, he suggests that unrestrained thermal curvature be calculated on the basis of the uncracked section, and that thermal continuity moments and secondary thermal stresses be calculated on the basis of moment of inertia associated with the expected distribution of cracking at service loads.

Mentes et al. (8) presented a method for estimating reduced stiffness due to thermal effects. The method is based on Branson's effective moment of inertia and an iterative procedure. Branson (9) has presented a method for calculating differential temperature effects in composite construction.

For nuclear power plant structures, ACI Committee 349 (10) recommends that analysis for mechanical loads be based on uncracked stiffness and that analysis for thermal effects be based on cracking due to mechanical loads. ACI 349 calculation procedures for thermal stresses are based on I_g for uncracked sections and I_{cr} for cracked sections rather than effective moment of inertia I_e as outlined in the ACI Building Code. For deflection calculations, the effective moment of inertia approach is generally more appropriate.

Figure 3 shows a two span one-way slab under gravity loads. The bending moment diagram for this slab indicates that only in the negative moment region does the moment exceed the cracking moment M_{cr} . If however, a temperature gradient is set up in the slab an additional bending moment is induced in the slab due to continuity. In this case the bending moment distribution due to thermal effects increases the positive moment in both spans. Summing moments due to gravity loads and thermal effects results in a bending moment diagram where the cracking moment M_{cr} is exceeded in the positive moment region. Calculation of effective moment of inertia should be based on maximum moment conditions in all regions of the slab.

In cases where stresses are developed in the member due to restraint to axial deformations, the induced stress due to axial restraint should be included in the calculation of cracking moment in a manner analogous to that for including the prestressing force in prestressed concrete beams as outlined for example in Ref. 11.

DESIGN EXAMPLES

The following examples illustrate the calculation procedures outlined above.

Example 1 - Simply Supported Vertical Wall Panel - Linear Temperature Gradient

$$\begin{aligned}\Delta t &= 40^{\circ}\text{F} \\ \alpha &= 0.0000055 \text{ in./in./}^{\circ}\text{F} \\ h &= 4 \text{ in.}\end{aligned}$$

(a) Single storey span: $L = 12 \text{ ft}$

$$\Delta = \frac{0.0000055 \times 40 \times 144^2}{4 \times 8} = 0.14 \text{ in.}$$

(b) Two storey span: $L = 24 \text{ ft}$

$$\Delta = \frac{0.0000055 \times 40 \times 288^2}{4 \times 8} = 0.57 \text{ in.}$$

Example 2 - Simply Supported Tee Section - Linear Temperature Gradient over Depth (Fig. 4b)

$$\begin{aligned}\Delta t &= 40^{\circ}\text{F} \\ \alpha &= 0.0000055 \text{ in./in./}^{\circ}\text{F} \\ h &= 36 \text{ in.} \\ L &= 60 \text{ ft (simply supported)} \\ \Delta &= \frac{0.0000055 \times 40}{36} \times \frac{720^2}{8} \\ &= 0.40 \text{ in.}\end{aligned}$$

Example 3 - Simply Supported Tee Section - Constant Temperature over Flange Depth (Fig. 4c)

$$\begin{aligned}I &= 69319 \text{ in.}^4 \\ n &= 26.86 \text{ in.} \\ \Delta t &= 40^{\circ}\text{F} \\ \alpha &= 0.0000055 \text{ in./in./}^{\circ}\text{F} \\ h &= 36 \text{ in.} \\ L &= 60 \text{ ft} \\ \phi &= \frac{\alpha}{I} \int_0^{36} t(y)b(y)(y-n)dy \\ &= \frac{\alpha}{I} \int_{33}^{36} 40 \times 96(y - 26.86)dy \\ &= \frac{88,013 \times 0.0000055}{69319} \\ &= 0.00000698 \\ \Delta &= \frac{\phi L^2}{8} = \frac{0.00000698 \times 720^2}{8} \\ &= 0.45 \text{ in.}\end{aligned}$$

Example 4 - Continuous One-Way Slab-Linear Temperature Gradient (Fig. 3)

$$\begin{aligned}
 h &= 6 \text{ in.} \\
 w_D &= 75 \text{ psf} \\
 w_L &= 100 \text{ psf} \\
 \Delta t &= 40 \text{ F} \\
 \alpha &= 0.0000055 \text{ in./in./}^\circ\text{F} \\
 L &= 16 \text{ ft.} \\
 A_s &= 0.465 \text{ in.}^2/\text{ft. top and bottom} \\
 I_g &= 216 \text{ in.}^4/\text{ft.} \\
 f'_c &= 4000 \text{ psi} \\
 f_y &= 60,000 \text{ psi} \\
 I_{cr} &= 59 \text{ in.}^4/\text{ft.} \\
 M_{cr} &= 2.85 \text{ ft.kips/ft.}
 \end{aligned}$$

$$(a) \quad D + L \quad \text{Maximum Negative Moment} = \frac{(w_D + w_L)L^2}{8}$$

$$\text{Maximum Moment} = 5.60 \text{ ft.kips/ft.}$$

$$\begin{aligned}
 I_e &= \left(\frac{M_{cr}}{M}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M}\right)^3\right] I_{cr} \\
 &= \left(\frac{2.85}{5.60}\right)^3 (216) + \left[1 - \left(\frac{2.85}{5.60}\right)^3\right] (59) \\
 &= 79 \text{ in.}^4
 \end{aligned}$$

$$\begin{aligned}
 \text{Maximum Positive Moment} &= \frac{9}{128}(w_D + w_L)L^2 \\
 &= 3.15 \text{ ft.kips/ft.}
 \end{aligned}$$

$$\begin{aligned}
 I_e &= \left(\frac{2.85}{3.15}\right)^3 (216) + \left[1 - \left(\frac{2.85}{3.15}\right)^3\right] (59) \\
 &= 174 \text{ in.}^4
 \end{aligned}$$

$$\text{Average } I_e = (79 + 174)/2 = 127 \text{ in.}^4$$

$$\begin{aligned}
 \Delta_{\max} &= \frac{wL^4}{185 EI} = \frac{(0.175)(16^4)(12^3)}{(185)(3600)(127)} \\
 &= 0.23 \text{ in.}
 \end{aligned}$$

- (b) $D + L + T$
At interior support, positive moment induced by temperature gradients is given by

$$M_T = \frac{3}{2} EI \phi \quad \text{where } \phi = \frac{\alpha \Delta t}{h}$$

- (i) Thermal moments based on I_g

$$\begin{aligned}
 M_T &= \left(\frac{3}{2}\right)(3600)(216)\left(\frac{0.0000055 \times 40}{6}\right) \\
 &= 7.13 \text{ ft.kips/ft.}
 \end{aligned}$$

Note that this moment exceeds the negative moment due to dead + live load.

At location of maximum positive moment due to dead + live load (i.e. $3/8$ L from end of span)

$$M_T = 3/8 \times 7.13 = 2.67 \text{ ft.k}$$

Max. positive moment for D + L + T

$$= 3.15 + 2.67 = 5.82 \text{ ft.k/ft.}$$

$$I_e = \left(\frac{2.85}{5.82} \right)^3 (216) + \left[1 - \left(\frac{2.85}{5.82} \right)^3 \right] (59)$$

$$= 78 \text{ in.}^4$$

$$\text{Average } I_e = (79+78)/2 = 78.5 \text{ in.}^4$$

Deflection due to D + L

$$\Delta_{\max} = \frac{(0.175)(16^4)(12^3)}{(185)(3600)(78.5)} = 0.37 \text{ in.}$$

i.e. Temperature gradient results in 60% increase in deflection under dead + live load

- (ii) Thermal moments based on I_e calculated for service loads

$$M_T = 7.13 \times \frac{127}{216} = 4.19 \text{ ft.kips/ft.}$$

$$M_T \text{ at } 3/8 \text{ L} = 1.57 \text{ ft.kips/ft.}$$

Therefore maximum positive moment for D + L + T = $3.15 + 1.57 = 4.72 \text{ ft.kips/ft.}$

$$I_e = 93 \text{ in.}^4$$

$$\text{Average } I_e = (79+93)/2 = 86 \text{ in.}^4$$

Deflection due to D + L

$$\Delta_{\max} = 0.33 \text{ in.}$$

SUMMARY AND CONCLUSION

Deflections caused by temperature change can have adverse effects on serviceability of concrete structures. Measures should be taken at the design stage to alleviate these effects, usually by providing adequate joint details to permit relative movement where required. Calculation procedures to estimate changes in stiffness and temperature-induced deflections are outlined in the report.

For uncracked members, effects of temperature can be included in deflection calculations in a relatively straightforward manner. For statically indeterminate systems after cracking, the deflections, stiffness, and temperature are inter-related, and an iterative procedure is required for a correct solution. However a simplified direct procedure is given in the report for calculation of temperature-induced deflections after cracking occurs.

Temperatures and temperature gradients for use in design are specified in several codes. Suggestions for selecting these design quantities are given in the report for cases where the temperatures to be considered are not specified by the governing code.

Some data are available in the literature concerning field measurements of temperatures in structures. Additional data would be of value in developing appropriate temperature gradients for standard design situations.

ACKNOWLEDGEMENT

The committee acknowledges the significant contribution made to this report by J.R. Libby, former Chairman of ACI Committee 435.

Conversion Factors - Inch-Pound to SI

1 in.	= 25.4 mm
1 lb (mass)	= 0.4536 kg
1 lb (force)	= 4.488 N
1 lb/sq in.	= 6.895 kPa
1 kip	= 444.8 N
1 kip/sq in.	= 6.895 MPa
1 in.-kip	= 0.1130 N•m

NOTATION

A	= cross-sectional area
b	= width of beam
D	= dead load
E	= modulus of elasticity
h	= depth of member, blacktop thickness
I	= moment of inertia
I_{cr}	= moment of inertia at cracked section
I_e	= effective moment of inertia
I_g	= moment of inertia of gross concrete section

k	= constant
L	= Live load, span length
M	= bending moment
M_{cr}	= cracking moment
n	= distance of neutral axis from bottom of beam
P	= axial load
$t, \Delta t$	= temperature difference
T	= effects of temperature
w_D	= uniformly distributed dead load
w_L	= uniformly distributed live load
y	= distance from bottom of beam
α	= coefficient of thermal expansion
Δ	= deflection
ϵ_a	= axial strain at centroid
ϵ_t	= strain due to temperature change, t
ϕ	= curvature
σ	= stress

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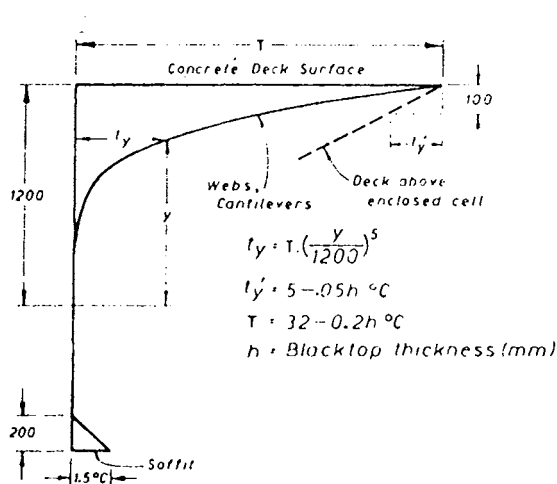


Fig. 1--New Zealand design thermal gradient for concrete bridges (from Ref. 7).

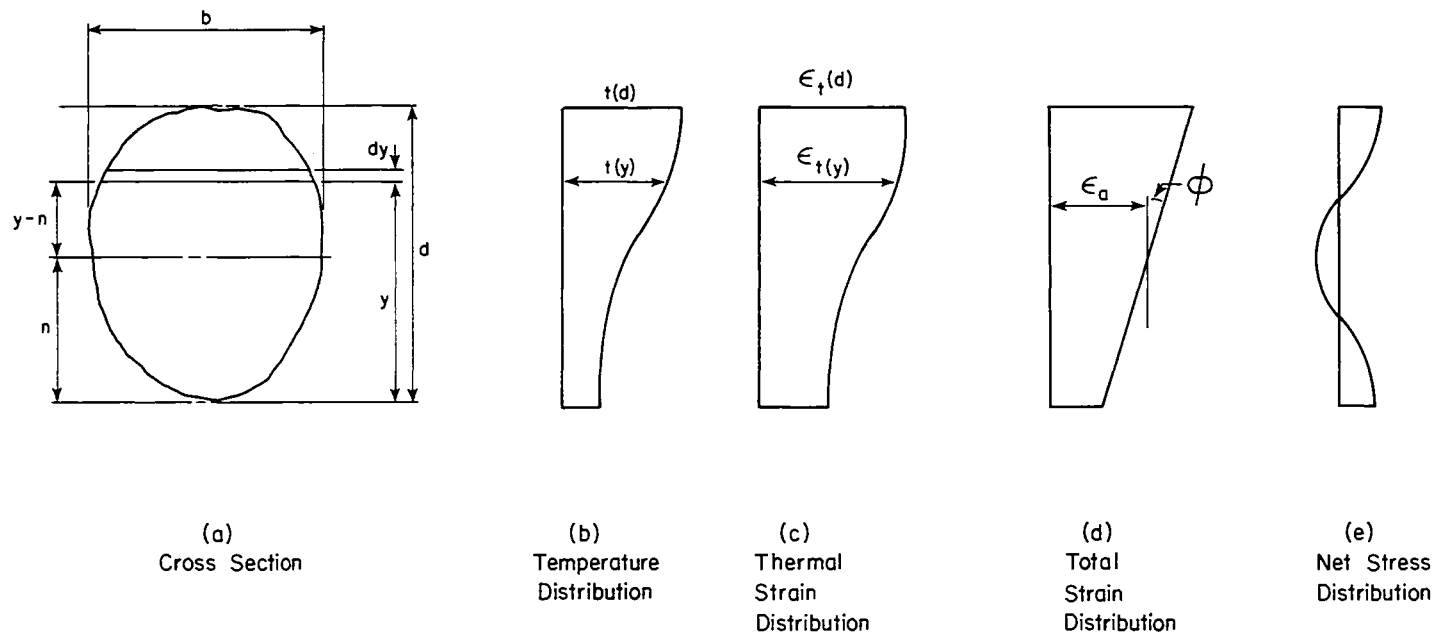


Fig. 2--Thermal stress and strain distribution on general cross section.

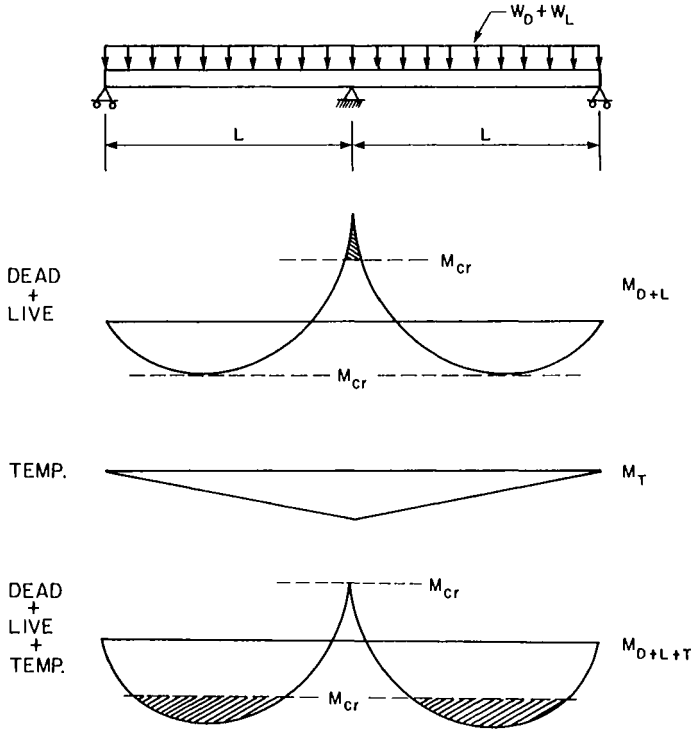


Fig. 3--Effect of temperature gradient on bending moments in two-span slab.

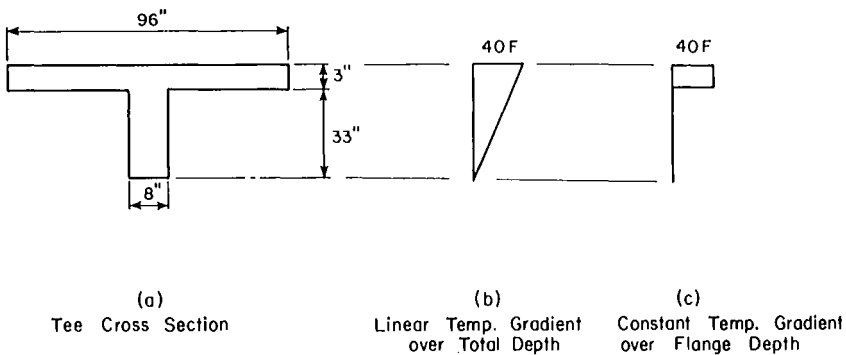


Fig. 4--Temperature gradients on tee cross section.

This report was submitted to letter ballot of the committee which consists of 19 members; 17 were affirmative, 2 were not returned. It has been processed in accordance with the Institute procedure and is approved for publication and discussion.

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

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**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

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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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Investigation D (Taylor, Heiman - five Sydney area buildings)

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Part II

FACTORS CONTRIBUTING TO DEFLECTION PROBLEMS IN TWO-WAY
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Factors Contributing to Slab Deflection Problems

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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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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., Sozen, 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 ($f_y \min = 70$ ksi). The deflection behavior of the slab was expressed by the equation:

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

- 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 $f_{y \min} = 50$ 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 ($f_y \min = 60$ 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.

References

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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.

Reference

- E.1. Börtemark, I., "Deformation of Gypsum Wallboard Partitions Erected Between Concrete Floors," Chalmers University of Technology, Division of Building Technology, 73:8, Göteborg, Sweden, 1973, 165 pp.

INVESTIGATION F

Summary

Tests were carried out on a panel on the fourth level of a building comprising five levels of flat plate floor, four bays wide and seven bays long (Fig. 7a). The column grid was 21 ft 3 in. x 18 ft 10 in. with columns 22 in. square. The slab was designed to be 9 in. thick to carry 120 psf live load. There were no column capitals or drop panels, but bent-up bars were provided to produce the desired shear capacity. The test panel was surrounded by other panels on all sides; however, two adjacent panels on the western side were only half size panels. Measurements of the slab depth indicated an average thickness of 9.57 in. The maximum span to depth ratio was therefore 26.7. Pre-mixed concrete was used, with a specified minimum compressive strength of 3 ksi at 28 days. At 108 days, when some tests were conducted, the average strength from four cylinders was 4.12 ksi. The experiments for short-term behavior, which were carried out on the test panel, were a series of loadings from zero to 140 psf uniformly distributed load. The south-west quadrant of the loaded panel was instrumented at various locations to measure various deformations. Deflection readings were taken during construction to record the effects of stripping of formwork from the slab, the sequence of propping, and the loading history of the slab. Deflection readings were continued into the service life of the structure. At the time of writing of the paper (Ref. F.1), the slab was 27 months old and the full service load had been in place for one year.

The first 14 months of the deflection record of the mid-point of the panel has been correlated with the loading history of the slab in Fig. 7b. The following observations may be worthy of note:

- i. The formwork was stripped after ten days and the initial deflection of the center of the panel was 0.079 in.
- ii. The panel deflected further when adjacent panels were unpropped and formwork was erected for the slab above. At this stage, the total deflection of the mid-point was 0.210 in. The magnitude of this deflection indicates the large loads carried by the slab during the construction of the building. Minor fluctuations in deflection occurred



- when the slab above was stripped and repped. When all the props were removed, the center of panel deflection was 0.181 in., more than twice the initial deflection, only 75 days after the slab had been poured.
- iii. The bricks which were to form the brickwork partitions were left in piles on the slab for two weeks to act as preload. By the time the bricklaying had commenced, the mid-point deflection was 0.201 in.
 - iv. After one year, the deflection of the center of the panel due to dead load was 0.311 in. - a factor of four increase on the initial deflection and equal to span/990.
 - v. Fourteen months later when live load was applied (25 psf to the southern half of the test panel, and adjacent panels to the south side of the test panel), the mid-point deflection was 0.339 in. After another 12 months with some additional live load (approximately 5 psf), the mid-panel deflection was 0.461 in. The loading was due to library books, and at this stage the shelves were about 60% full. Thus, it was unlikely that the permanent live load would exceed 60 psf.
 - vi. A method of deflection prediction proposed by Taylor (Refs. D.1 and D.3) was used to estimate deflections. Figure 7c shows the comparison of measured and predicted deflections for the slab after 12 months under dead load. The approximate method of long-term deflection prediction gave excellent agreement with the experimental results.

Reference

- F.1. Jenkins, B.R., "Tests on a Flat Plate Floor", Civil Engineering Transactions, The Institution of Engineers Australia, Vol. CE 16, No. 2, 1974, pp. 164-167.

INVESTIGATION G

Summary

Slab deflections in a multistory flat plate building were recorded at 175 bays on 13 floors of the upper part of the structure one year after casting. Lateral force resistance of the flat plate building was supplied by a stiff system of beams and closely spaced wide columns. The design was in accordance with the 1971 ACI Code. The center-to-center slab spans were 21.6 and 22.4 ft. Three sources of information were used to obtain slab thickness and the variation in slab thickness. They included

TABLE 1 - MATHEMATICAL MODEL USED IN INVESTIGATION G

STRUCTURE

Center-to-center span, $\ell = 22$ ft

Clear span, $\ell_n = 0.9\ell$

Slab thickness, $t = 7 \frac{1}{4}$ in., standard deviation =
1/4 and 1/3 in.

Reinforcement, At columns, $A_s = 5.0$ sq in./strip
Elsewhere, $A_s = 1.8$ sq in. strip

CONCRETE

Lightweight, 110 lbs/cu ft

28-day strength, $f'_c = 4000$ psi, coeff. of variation = 10%

Ultimate creep coefficient, $C_u = 2.35$

Ultimate shrinkage strain, $(\epsilon_{sh})_u = 800$ micro-in./in.

Other properties and variation in properties with time in
accordance with ACI Committee 209 recommendations.

LOADS

Weight of slab, 67 psf

Other dead loads, 20 psf

Construction live load, 50 psf

Weight of forms, 15 psf; Reshores, 5 psf

Rate of Construction, 2 floors/week

Case 1, 8-floor supporting assembly, $N = 8$, $w_{max} = 144$ psf

Case 2, 3-floor supporting assembly, $N = 3$, $w_{max} = 157$ psf

Case 3, 2-floor supporting assembly, $N = 2$, $w_{max} = 164$ psf

Case 4, Combination of deflections, 80% from Case 2 and 10%
each from Cases 1 and 3.

Maximum construction dead and live loads, w_{max} in accordance
with Refs. 1 and 2, Investigation G.

Long-term effects, one-year multiplier = 4.2 (Ref. 2,
Investigation G, for $\ell/t = 35$)

direct measurements at floor openings and holes, the quantity of concrete used per floor, and level readings recorded for the formwork on the day before casting and for the top of the concrete on the day after casting. It was determined on the basis of these measurements that the level of workmanship was represented by an average thickness of 7.25 inches and a standard deviation of $1/4$ to $1/3$ in.

The supporting assembly which carried the fresh concrete reportedly consisted of three sets of forms with reshores extending 5 to 7 floors below ($N = 8$ to 10). A two floor per week schedule was maintained. The sequence of stripping and reshoring had a significant influence on long-term deflections. Some areas were reportedly stripped and reshored half a bay at a time. On most occasions stripping reportedly occurred over large areas or entire floors prior to reshoring. Instances of stripping of large areas on the day of casting or while the casting operation was in progress were also reported. Construction photos showed examples of all the above construction procedures. They also showed that the reshoring was not closely spaced, it did not line up from floor to floor consistently, or was omitted. If forms are repeatedly stripped over large areas prior to reshoring, the slab being stripped would receive the maximum construction load at the time of form stripping. Thus, even if the supporting assembly consisted of 8 or more slabs, the construction loads absorbed by the slab being stripped would be those attributable to a 3-slab supporting assembly, or $N = 3$. If stripping occurred while the new slab was still wet, the supporting assembly would consist of two slabs, $N = 2$. The reported prevailing construction procedure was felt to be equivalent to a 3-floor supporting assembly, $N = 3$. Instances of supporting assemblies of $N = 8$ and $N = 2$ were felt to be infrequent.

Materials for partitions, ceiling, the building exterior, etc. were delivered and stored on the intended floors beginning about $1\frac{1}{2}$ to 2 months after casting. The estimated equivalent load during storage was 25 to 30 psf. After the installation of finishes, the estimated load was 12 to 15 psf. The additional service dead load contributing to creep was assumed to be 20 psf.

Elevations of slabs were obtained on the day after casting and one year after construction. Net deflections were obtained by comparing the two sets of level readings. Differential movement at the columns was accounted for by referencing to the recorded elevations at the columns, and the deflections discussed were the net measured deflections produced by loads or shrinkage. The net measured one-year deflections ranged from 0.53 to 2.16 in. They are shown by the dashed line in Fig. 8. They averaged 1.35 inches with a standard deviation of 0.29 inches and a coefficient of variation of 21.2 percent. In 90 percent of the cases deflections exceeded 1 in. In 10 percent of the cases they exceeded 1.72 in.

The calculated deflection for the 7 1/4 in. slab loaded to the service dead loads of $67 + 20 = 87$ psf at a 28-day strength $f'_c = 4000$ psi was 0.21 inches. The ratio of the measured one-year deflection to the above value averaged 6.4 and ranged from 2.5 to 10.3. Long-term plus short-term deflections were calculated by applying the procedures developed in Refs. G.1 and G.2. Attempts to compare measured and calculated deflections for specific bays were unsuccessful. There was considerable uncertainty about the thickness of the slab in any one bay. The construction load for any one bay was also uncertain. For this reason, a mathematical model consistent with all observations was developed. The results from the mathematical model were then compared with the measured deflections (Table 1 and Fig. 8).

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MISCELLANEOUS ITEM

Summary

City engineers of Edinburgh, Scotland, declared unsafe a seven-story, five-year-old reinforced concrete building in the city because of cracks found in the ground floor slab that had deflected 5 in.

The engineers ordered the building vacated until the owner, Scottish Life Assurance Society, could decide whether to reinforce or demolish the structure. The 50,000-sq-ft, \$1.4-million structure housed the Scottish Stock Exchange.

The 5-in. deflection was discovered when a ground-floor tenant decided to lift the floor to check for failure after the slab showed signs of deflection. Engineers found that the central slab spanning 32 ft over the building's parking area was completely cracked with hairline fractures from 1.5 to 3 in.

long. The upper floors had deflected from 1.5 to 3 in. In addition, two ground floor columns were found to have soft spots.

One engineer blamed the use of a certain type of local aggregate in the concrete mix for the deflections. The aggregate had a high shrinkage rate, which distorted the original structural dimensions, according to the engineer.

Studies being conducted at the time of this report (December 1971) by Edinburgh consulting engineer Kinnear and Gordon, with a design check by Ove Arup and Partners, London, were expected to determine the seriousness of the defects and the best method of correcting them. No cost of repairs was available. The floor slabs had been braced with steel supports.

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Engineering News Record, Vol. 187, No. 25, Dec. 16, 1971, p. 19.

Part II

FACTORS CONTRIBUTING TO DEFLECTION PROBLEMS IN
TWO-WAY REINFORCED CONCRETE SLABS

Factors contributing to problematic slab deflections are examined in this part of the report. Sequences of form removal and reshoring, directed toward avoidance of large slab deflections, are outlined.

FACTORS CONTRIBUTING TO SLAB DEFLECTION PROBLEMS

The following is a list of factors contributing to excessive long-term slab deflections. The list largely follows a compilation by Taylor and Heiman.¹

1. Formwork has not been cambered in cantilevers and large interior panels, so that even early deflections have become obvious.
2. Slabs have been supported by props bearing on sole plates that were of inadequate size to prevent appreciable settlement into the ground surface and hence induced slab deformation before stripping.
3. Construction loading from propping or the storage of materials during the early life of the slabs has been severe enough to cause extensive slab cracking and hence loss of stiffness.
4. Top reinforcement at supporting elements has been pushed down during slab construction, substantially reducing its effective depth, and hence reducing the contribution made to slab stiffness by continuity at supports.
5. Column concrete has sometimes been carried too high, beyond the slab soffit. If this extra concrete is not chipped before the slab is cast, the resulting cold joint in the support region actually reduces effective slab depth where it is needed.
6. Some coarse aggregates which are dimensionally unstable have at times produced concrete with abnormally high shrinkage characteristics.
7. Curing has in many instances been inadequate, and has sometimes led to insufficient strength development and excessive shrinkage cracking.

Items 4 and 5 above are of surprisingly common occurrence. Quite often the top steel over supports is not securely held in place and is displaced towards the neutral axis, greatly reducing the stiffness over the support. If the effective depth is 7 in. instead of 8 in., as has been observed, the stiffness is reduced by 23%, and the member tends to behave as a simply reinforced one.²

Discussion in the rest of this report will focus on Item 3 above. While Taylor and Heiman¹ mentioned extensive slab cracking and loss of stiffness, phenomena that are difficult to quantify, the following discussion will be in terms of higher initial deflections and greater creep caused by high construction loading of concrete at an early age which is perhaps the most important factor to consider.

LOADS DURING CONSTRUCTION

Structural formwork and its support system deserve careful consideration in two respects: (1) from the point of view of loads which may be applied to the formwork and its props, and (2) from the point of view of loads which the formwork and the props may apply to the structure. The second aspect is of primary interest in this paper.

In the construction of multistory buildings with reinforced concrete floor slabs, a step-by-step sequence of operations is employed. The sequence is comprised of: setting up shoring on the most recently placed floor, forming the next floor, setting the reinforcement, and concreting the slab. Since the floor below the one being concreted will usually be between seven and fourteen days old, it is common practice to leave formwork props in place between that floor and one or two floors below it.

It is convenient in discussing construction loads to express them as a factor times the sum of self-weight of the floor and the dead load of the formwork. The term "floor loading ratio" is used for this factor.

Table 2, reproduced in a slightly abbreviated form from Ref. 3, indicates that all investigators on the subject agree that floor loading ratios during construction usually exceed values of 2. This theoretical conclusion has been verified by Agarwal and Gardner⁹ using shores instrumented with electrical resistance strain gages.

The analyses of floor loads in Table 1 are based on the assumptions that:

1. The slabs behave elastically,

2. Initially the slabs are supported from a completely rigid foundation, and
3. The props supporting the slabs and formwork may be regarded as a continuous uniform elastic support, the elastic properties of which may be expressed by a coefficient K , where K = load intensity that produces unit deformation of the support.

Grundy and Kabaila⁵ assumed K to be infinite. Beresford⁶ used infinite as well as various finite values of K , and found that results were not appreciably affected. Grundy and Kabaila⁵ carried out their analyses assuming constant flexural stiffness for all connected slabs, as well as flexural stiffness increasing with age. It was found that the error introduced by assuming equal relative stiffness for the floors is not appreciable. Blakey and Beresford⁷ recommended a stepped sequence of construction in a system of floors and shores as a means of controlling the construction loads imposed on both the slabs and the props. The advantage of this method of construction lies in that a young slab is given more time to develop adequate strength before the application of construction load from the casting of a new slab directly above the segment.

Table 2 is for construction operations using multiple sets of forms. Economic considerations usually necessitate the removal of formwork as soon as possible for re-use. This necessity has given rise to the widespread practice of reshoring. Reshoring is installed after the shores under a slab and the formwork held by them are removed and after the slab assumes a natural deflected shape. At the time of installation reshores carry no significant load.

Taylor¹⁰ recommended a method to release and tighten shores under a floor slab as a means of reducing construction loads on the connected slabs. Taylor's method is the same in principle as stripping and immediate reshoring of a slab. Marosszeky¹¹ described complete release and reshoring of a floor slab, such that the floor carried its own dead weight at a time $(T-1)$ days, where T is the construction cycle of floors. This reshoring technique produces less construction load on the supporting slabs and props, in comparison with using undisturbed shores. Table 3, adapted from Ref. 12, shows absolute maximum and converged maximum load ratios on slabs and supporting props for various combinations of levels of shores and reshores.

It should be noted that construction live loads are not considered in Table 2 or 3. Lasisi and Ng¹² have shown that the consideration of 50 psf construction live load, with the assumption that such load no longer acts on the freshly poured slab beyond the end of the casting day, increases the absolute maximum load ratio from 1.83 to 1.99 (an increase of 9%) in the case of two levels of shores and one level of reshores.

Table 3 indicates that the use of two levels of shoring and 1 level of reshoring rather than 3 levels of shores, reduces the absolute maximum load ratio from 2.36 to 1.83, and the converged maximum load ratio from 2.00 to 1.78. This is advantageous in most situations, although with reshoring, the maximum load ratios come into play at an earlier age than with shoring. Table 4 is a compilation of construction loads for Floor 3 which experiences the absolute maximum load ratio with three levels of shoring or shoring plus reshoring. The construction loads are compared with the design service loads in the table. It is clear that the construction loads are more critical than the design loads. Also, importantly, the construction loads act on concrete that has not attained the age at which it is supposed to experience the design service loads.

PROPERTIES OF CONCRETE AT EARLY AGES

During the construction of a multistory building, a concrete floor slab may be required to carry its own weight at 5 days of age or earlier, and substantially higher loads well before it is 28 days old. The property of young concrete that is of particular interest here is the modulus of elasticity.

According to ACI Committee 209,¹³ the effect of age of concrete (at the time of loading) on the modulus of elasticity of concrete is accounted for when strength of concrete at the time of loading, rather than at 28 days, is inserted into the modulus of elasticity expression given in the ACI Code¹⁴ (Fig. 9):

$$E_{ct} = 33 w^{1.5} \sqrt{f'_{ct}} \quad (2)$$

where E_{ct} is the time-dependent modulus of elasticity of concrete in psi;
 w is the unit weight of concrete in pcf; and
 f'_{ct} is time-dependent concrete strength in psi.

ACI Committee 209¹³ has also recommended the following expression for the time-dependent strength of moist-cured (as distinct from steam-cured) concrete using Type I cement (Fig. 10):

$$f'_{ct} = \frac{t}{4.00 + 0.85t} f'_c \quad (3)$$

where t is the time in days from casting up to loading, and f'_c is the 28-day compressive strength of concrete.

The validity of Eq. (3) at very early ages may be open to question. A fair amount of detailed information on the compressive strength of concrete at early ages is now available, partly as a result of the RILEM symposium on the subject that was held in Paris in 1981^{15,16}, although such information is probably still far from sufficient.

Eq. (2) for normal weight concrete, when converted to SI units and rounded off, becomes:

$$E_{ct} = 5000 \sqrt{f'_{ct}} \quad (4)$$

where E_{ct} , f'_{ct} are in N/mm^2 . Fig. 11 shows a comparison of Eq. (4) with the experimental information currently available on the modulus of elasticity of concrete at early ages.¹⁵ Concrete strengths of 1 N/mm^2 (145 psi) and less are encountered only during the first few hours after pouring of concrete, and are not of interest here. In the range of practical interest, the ACI equation relating modulus of elasticity and strength agrees quite favorably with the trend of experimental results.

CREEP OF CONCRETE LOADED AT EARLY AGES

For a given concrete the amount of creep depends to a great extent on the age of the concrete at the time of loading. Fig. 12 shows the relationship between creep and age at loading, as developed by CEB,¹⁷ using available information from many tests. The coefficient α_{age} relates the creep for any age at loading to the creep of a specimen loaded at the age of 28 days. The 28-day creep is used as a basis of comparison, the corresponding α_{age} being equal to 1.0.

Fig. 12 also depicts the following relationship between creep and age at loading, recently developed by Fintel, Ghosh and Iyengar:¹⁸

$$\alpha_{\text{age}} = 2.3 t_{\text{LA}}^{-0.25} \quad (5)$$

where t_{LA} is the age of concrete at the time of loading, in days. The form of Eq. (5) is as suggested by ACI Committee 209.¹³ Eq. (5) gives better correlation with the CEB mean curve than the corresponding equation suggested by the above Committee. Fig. 12 also shows comparison with a few experimental results.^{19,20} According to Eq. (5), the creep of concrete loaded at 7 days is 41% higher than that of concrete loaded at 28 days of age.

Another important aspect of creep of concrete needs to be pointed out. Fig. 13 schematically shows the strain history of a concrete specimen, stored in air at near 100% relative humidity to eliminate shrinkage, and subjected to a sustained axial stress which was removed at 120 days from the time of loading. On removal of loading, the strain decreases immediately by an amount equal to the elastic strain at the given age, generally lower than the elastic strain on loading. This instantaneous recovery is followed by a gradual decrease in strain, called creep recovery. The shape of the creep recovery curve is similar to that of the creep curve, but the recovery approaches its maximum value much more rapidly. The reversal of creep is not complete, so that any sustained application of load, even over only a day, results in a residual deformation.²¹

CONTROL OF SLAB DEFLECTIONS

Probably the surest way of controlling deflections is by carefully sequencing the operations of shore removal and reshoring. With a column layout like the one shown in Fig. 14, stringers would normally be run in the short direction at about 4 ft o/c, supported on shores. Ribs or purlins will then be run in the orthogonal direction, also at about 4 ft o/c, supported on the stringers. Finally, the 4 ft x 8 ft plywood sheets would be supported on the ribs.

What is suggested here on the basis of successful experience with flat plate buildings in the New York area is that alternate plywood sheets in both the long and the short direction (shown shaded in Fig. 14) be supported directly by extra shores (indicated by 'x' on Fig. 14). These latter shores (attached to the plywood, rather than to the stringers, as the other shores are) may be installed at the time the other shores are installed, or just before shore removal.

When the time comes for removal of formwork, a day or two after casting or when concrete strength reaches a certain minimum value, the regular shores, the stringers, the purlins, and the plywood sheets that are not directly held by the extra shores, can all be removed. However, before the extra shores and the plywood sheets held by them are removed, reshores should be installed at about 8 ft o/c directly to the concrete slab. The extra shores or the so called permanent shores should be removed only after the reshores have been installed.

The above scheme of removing the shores and installing the reshores does not permit more than 8 ft of slab span to be left unsupported at any time until the slab is sufficiently mature. With such short unsupported slab spans, slab deflections under usual circumstances cannot assume disturbing proportions, however high the loading may be during construction, and however immature the slab may be when it is called upon to support those loads.

The above consideration applies with the use of flying forms also. Fig. 15 shows a reinforced concrete slab supported on reinforced concrete columns spaced at 20 ft o/c in both directions. With such a column layout, an 18 ft wide form table would normally be used. However, in that case, as soon as the flying form is removed, 18 ft of slab span would be left unsupported. If deflections are of concern, two 8 ft wide form tables, with a filler strip of formwork in between, may be used instead. With the narrower form tables, even when they are removed, no more than 8 ft of slab span would be left unsupported. Admittedly, two 8 ft wide form tables with a filler strip of formwork are significantly more expensive than a single 18 ft wide form table. The added expense has to be weighed carefully against any advantage that is to be gained in terms of reduced deflections.

SUMMARY AND CONCLUSIONS

The sometimes excessive long-term deflection of flat plate and other two-way reinforced concrete slab systems has been a recurring problem through the years. This report contends that long-term deflection problems in two-way reinforced concrete slab systems are more often than not caused by construction deficiencies. A common example of construction deficiency is the incorrect placement of top reinforcement which plays an important role in minimizing long-term deflections. The aspect of construction that is given primary attention in this paper has to do with the sequence of shoring, form removal, and reshoring. It is pointed out that the construction loads on a slab system may be higher than the full service loads for which it is designed. These loads, acting on immature concrete which has a low modulus of elasticity, cause large immediate deflections. These deflections increase with time due to creep. Concrete loaded at an early age suffers larger creep deformations than concrete loaded at later ages. Although parts of the construction loads may be sustained for only a few days and then be removed, they still have an adverse effect on slab deflections, since creep is not a totally reversible phenomenon.

The loads on a slab during construction can be controlled by adjusting the number of levels of shores and reshores that are used (Table 3). However, in general, the loads can be minimized only by allowing the slab to deflect at an earlier age.

Some direct measures aimed at eliminating long-term deflection problems of reinforced concrete slab systems are outlined in this report.

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Table 2: Floor loading ratios³

Author	Maximum value, converged values in brackets				Comment
	m = 2	m = 3	m = 4	m = 5	
Nielsen ⁴	2.17 2.0Δ	2.28x 2.56+			Values for floor level 2 only x Timber props + steel props (n = 1) ΔObserved
Grundy Kabaila ⁵	2.25 (2.00)	2.36 (2.00)	2.43 (2.00)		(n = 5)
Beresford ⁶	2.25	(2.00)* (2.06)*2.35 (2.32)	2.45	2.50	*Obtained for rapid hardening, normal & slow maturing concretes respectively (n = 5)
Blakey Beresford ⁷	2.25 2.25+	2.3			+Stepwise construction
Beresford ⁸		2.2 1.5Δ			(n = 4) ΔObserved

m = number of levels of shoring used

n = time in days for removal of lowest levels of shores after concreting top floor

Table 3: Theoretical maximum load ratios on floor for various shore/reshore combinations.¹²

Shore + reshore	Absolute maximum load ratio	Converged maximum load ratio
1 + 1	1.50	1.50
1 + 2	1.34	1.34
1 + 3	1.25	1.25
1 + 4	1.20	1.20
2 + 0	2.25	2.00
2 + 1	1.83	1.78
2 + 2	1.75	1.67
2 + 3	1.61	1.60
3 + 0	2.36	2.00
3 + 1	2.10	1.87
3 + 2	1.97	1.80

Table 4: Comparison of construction loads with service loads

Construction loads, psf				Service loads, psi		
3 levels of shores		2 levels of shores plus 1 level of reshores				
8 in. slab		100	8 in. slab		100	
Formwork		10	Ceiling & Mech		15	
			Partitions		20	
Subtotal		110	Live load		50	
Load Ratio	1.00 at 5 days	110	1.00 at 5 days	110	Total (after 28 days)	185
	1.34 at 7 days	147	1.00 at 7 days	110		
	1.45 at 12 days	160	1.50 at 12 days+	165		
	1.78 at 14 days	196	1.83 at 14 days	201		
	2.03 at 19 days	223*	1.00 at 19 days	110		
	2.36 at 21 days	260				
	1.00 at 26 days	110				

*Deflection not restrained beyond 19 days

⁺Allowed to deflect under these construction loads at 12 days.
Further deflections partly restrained for the next 7 days.

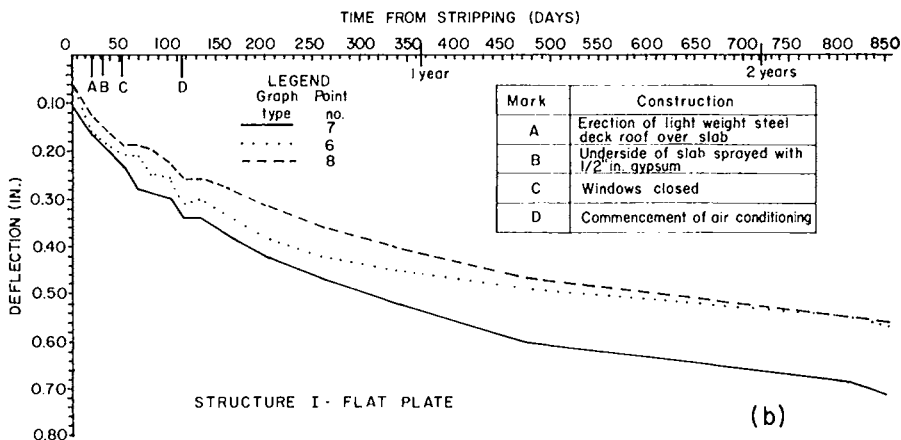
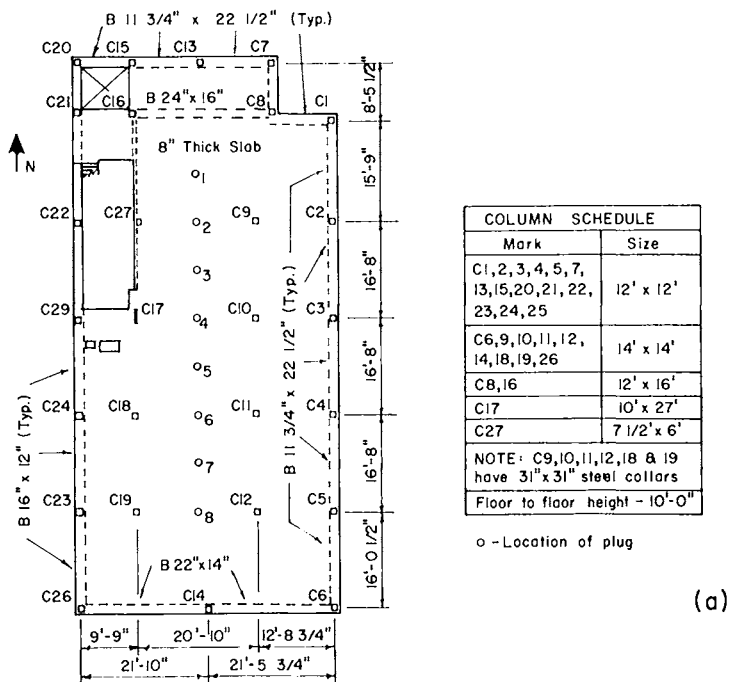


Fig. 1--Building studied and measured deflections - Structure I, Investigation D.

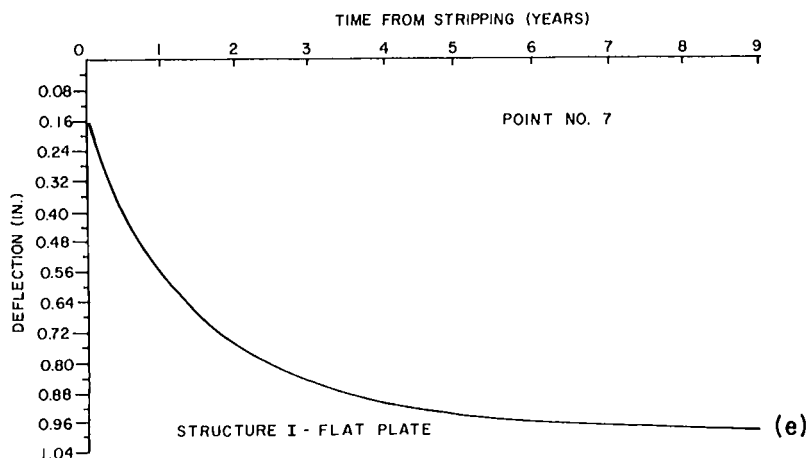
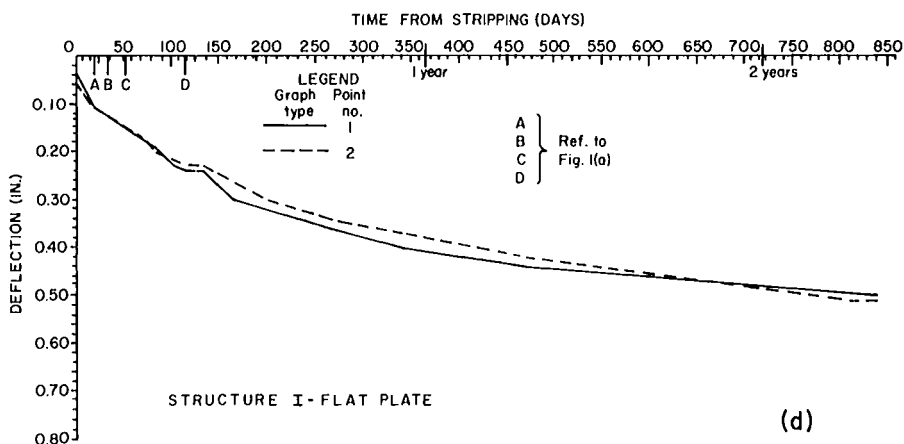
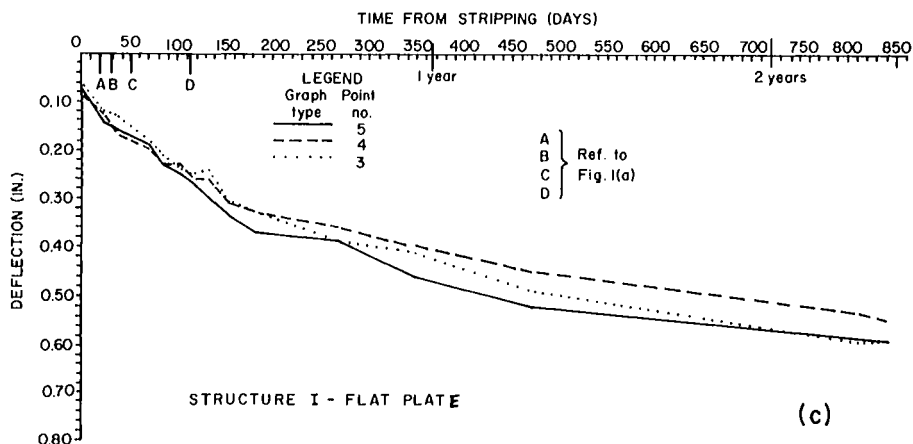


Fig. 1--Building studied and measured deflections - Structure I, Investigation D (continued).

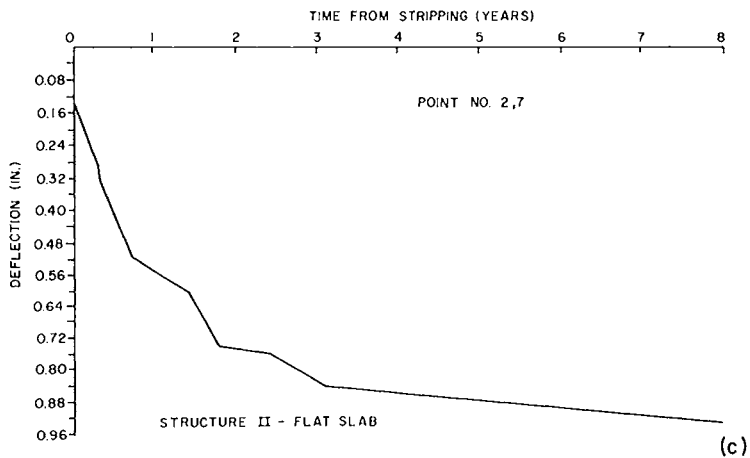
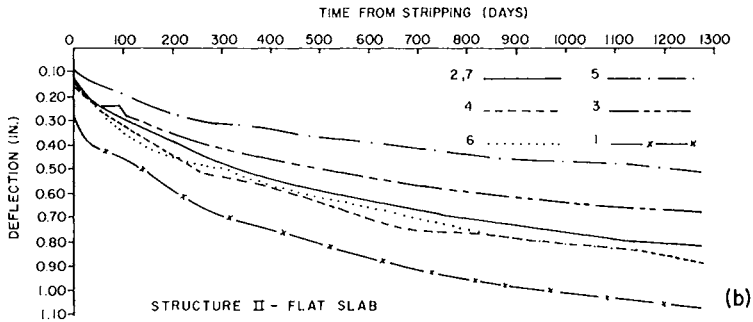
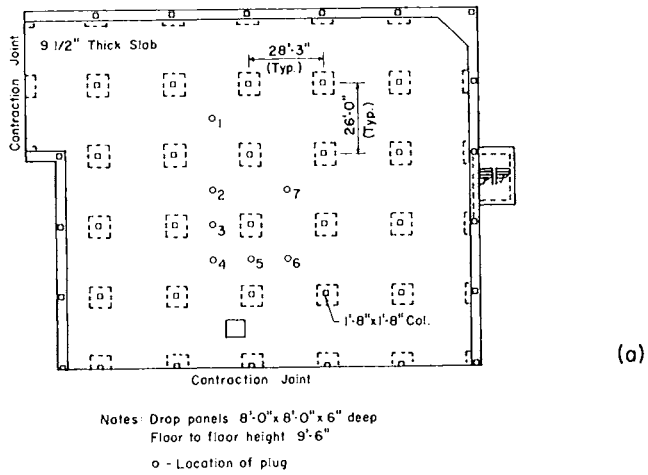


Fig. 2--Building studied and measured deflections - Structure II, Investigation D.

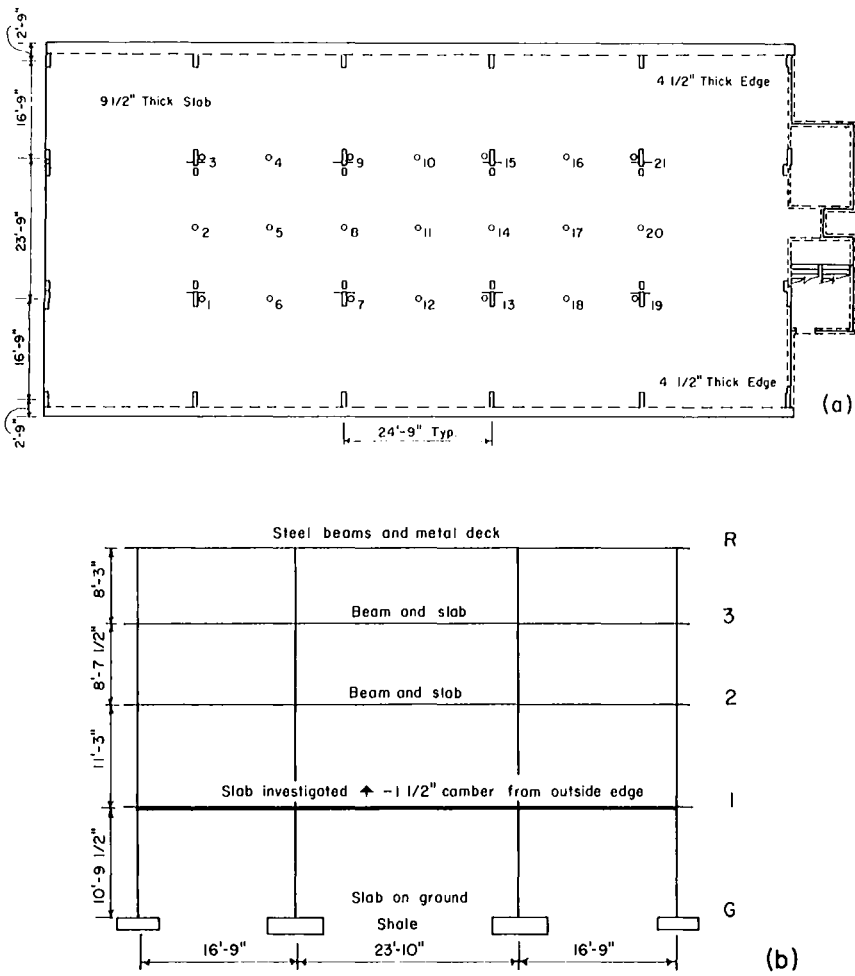


Fig. 3--Building studied and measured deflections - Structure III, Investigation D.