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ACI Code Deflection Requirements—Time for a Change?

by K. B. Bondy, FACI

Synopsis: This paper critically examines the deflection criteria in Chapter 9 of the current ACI Building Code, ACI 318-02, with a particular focus on two-way non-prestressed slabs. The relationship between criteria based on deflection computations and arbitrary minimum thicknesses, which are independent of loading and concrete strength, are scrutinized. A numerical example is presented in which it is demonstrated that current code criteria can lead to unsatisfactory performance in heavily loaded slabs. Recommendations are made for changes to improve code deflection criteria.

Keywords: code; computations; deflection; slabs

AUTHOR'S BIOGRAPHY

ACI Fellow ***Kenneth B. Bondy*** is a consulting structural engineer in West Hills, CA. He is a member of ACI Committee 318, Structural Concrete Building Code; Joint ACI-ASCE Committee 423, Prestressed Concrete; and the ACI Board Committee on Responsibility in Concrete Construction. An engineering graduate of the University of California at Los Angeles, he has specialized in the design and construction of concrete buildings for more than 41 years.

INTRODUCTION

The current ACI *Building Code Requirements for Structural Concrete* (ACI 318-02) controls flexural deflection in reinforced concrete members in Section 9.5. Section 9.5.2 addresses deflections in non-prestressed one-way construction, 9.5.3 in non-prestressed two-way construction, 9.5.4 in all prestressed concrete construction, and 9.5.5 in composite construction. While many of the comments in this paper apply philosophically to all four of these code sections, attention will be focused herein only on the requirements for non-prestressed two-way construction in Section 9.5.3, and more specifically, on deflection criteria for two-way slabs without interior beams.

DEFLECTION CRITERIA FOR NON-PRESTRESSED TWO-WAY SLABS WITHOUT INTERIOR BEAMS

Two-way slab systems without interior beams are addressed in Sections 9.5.3.1, 9.5.3.2, and 9.5.3.4. In 9.5.3.1 the code states, *"The thickness of slabs without interior beams spanning between the supports on all sides shall satisfy the requirements of 9.5.3.2* ***or*** *9.5.3.4."* [emphasis by author]

Section 9.5.3.2 establishes minimum thicknesses for two-way slabs in Table 9.5(c) – <u>MINIMUM THICKNESS OF SLABS WITHOUT INTERIOR BEAMS</u>. The minimum slab thicknesses are expressed in the form of the longest slab span ℓ_n divided by a numerical coefficient. The coefficient varies as a function of the presence of drop panels (as defined in Chapter 13), interior or exterior panels, and for exterior panels, the presence of an edge beam.

Section 9.5.3.4 permits the use of slab thicknesses less than the minimum thicknesses specified in Table 9.5(c) if it is "*...shown by computation that the deflection will not exceed the limits stipulated in Table 9.5(b).*" Table 9.5(b) establishes limits on computed deflections (expressed as a span ℓ divided by a numerical coefficient – more later on the definition of the term ℓ) for roof or floor members, differentiating between members that either are or are not "*...supporting or attached to nonstructural elements likely to be damaged by large deflections.*" For roofs or floors ***not*** supporting or attached to such vulnerable non-structural elements the deflection limits apply only to the immediate deflection due to live load. For roofs or floors that ***are*** supporting or attached to non-structural elements, the deflection limits apply to the "*...part of the total deflection occurring after attachment of nonstructural elements.*"

Section 9.5.3.4 also includes guidance on the appropriate concrete modulus of elasticity and slab moment of inertia to be used in calculations for instantaneous deflections, and cites the use of Section 9.5.2.5 for computing additional long-term deflections resulting from creep and shrinkage.

It is emphasized that code deflection requirements are satisfied by conformance to ***either*** Table 9.5(b) ***or*** Table 9.5(c). Conformance to both is not required. It is also emphasized that the minimum thickness requirements of Table 9.5(c) are ***independent of loading*** on the slab. Any two-way non-prestressed slab, regardless of superimposed load, satisfies code deflection requirements if it satisfies the minimum thickness requirements of Table 9.5(c). Calculations are only required to justify the use of slabs with thicknesses less than those specified in Table 9.5(c). No calculations are required for slabs with thicknesses equal to or greater than those specified in Table 9.5(c), regardless of slab loading. This is emphatically supported by ***PCA Notes on ACI 318-02***[1] which states on page 10-10, *"Deflections of two-way slab systems with and without beams, drop panels, and column capitals need not be computed when the minimum thickness requirements of 9.5.3 are met."*

The author questions the wisdom of basing slab deflection requirements on criteria that are completely independent of applied load. A two-way slab system spanning 30 feet (9.15 m) with a thickness of 12 inches (305 mm) (ℓ/30) satisfies code deflection criteria regardless of whether it carries a superimposed load of

50 psf (2.4 kN/m^2) or 500 psf (24 kN/m^2). Further, the slab would satisfy code deflection criteria even if calculations (which are not required) clearly show that the deflection criteria in Table 9.5(b) are violated. This does not seem rational. For heavily loaded slabs this approach can and has resulted in excessive deflections causing significant damage to supported elements. Yet these obviously inadequate designs are apparently in conformance with the code.

HISTORY OF MINIMUM THICKNESS REQUIREMENTS

Minimum thickness requirements for deflection control have appeared in the ACI code for more than forty years, and they have always been permitted to supersede criteria based on computed deflections. Like many other empirical or arbitrary criteria that have been in the code for a very long time, they probably were intended to provide a conservative solution in lieu of complicated calculations. This philosophy was certainly valid prior to the age of computers, which make complicated calculations transparent. The use of arbitrary minimum requirements are valid, however in the author's opinion, they should not permit the use of less restrictive criteria than required by calculations, even if the calculations are complicated. The commentary to 9.5.2.3 states:

> "***R9.5.2.3*** *– The minimum thicknesses in Table 9.5(c) are those that have been developed through the years. Slabs conforming to those limits have not resulted in systematic problems related to stiffness for short- and long-term loads.* ***These limits apply to only the domain of***

previous experience in loads, environment, materials, boundary conditions, and spans." [emphasis by author]

Unfortunately, the *"...domain of previous experience in loads..."* is not stated quantitatively. If it were, the author's criticisms would be greatly diminished. As the code is presently written, the minimum thickness requirements in Table 9.5(c) are, for all practical purposes, independent of applied load.

A PRACTICAL EXAMPLE

An example of a common type of construction in which the current code deflection criteria can lead to inadequate designs is the transfer slab or "podium slab", used in a type of residential building commonly called a "subterranean apartment building". In these buildings, two-way reinforced concrete slabs, prestressed and non-prestressed, are used to support two to four stories of wood-framed apartment buildings above the slab and form a ceiling for below-grade parking under the slab. Many thousands of such buildings have been constructed throughout the country. They are particularly prevalent in California. The slab is supported on reinforced concrete columns (generally round, 12-16 inches (305-407 mm) in diameter) and perimeter masonry retaining and bearing walls. Typical maximum bay sizes are roughly (27-29 ft)x(29-31 ft) [(8.3-8.8 m)x(8.8-9.5 m)], with the smaller dimension representing three parking stalls between columns and the larger dimension representing the span across the driving aisle. Occasionally only two parking stalls are provided between columns in which case the smaller bay dimension is about 19-21 feet (5.8-6.4 m). Slab thickness normally ranges between 12 and 16 inches (305 and 406 mm) for non-prestressed slabs and 10 to 14 inches (254 to 356 mm) for post-tensioned slabs. Typically a small square or rectangular shear cap is used at each column, with side plan dimensions of 3 to 9 feet (0.92 to 2.75 m) and 8-12 inches (203-305 mm) thick below the slab soffit. Design slab concrete strengths (normal weight) are typically 3,000-4,000 psi (21-28 MPa) for non-prestressed slabs and 4,000 psi (28 MPa) for post-tensioned slabs. Superimposed unfactored dead loads on these slabs range from 100-200 psf (4.8-9.6 kN/m^2) depending on the number of floors and the type of roof. Superimposed unfactored live loads range from about 80-150 psf (3.8-7.2 kN/m^2). Figure 1 shows a generalized cross-section of a subterranean apartment building with typical floor loadings.

The following numerical example is representative of numerous non-prestressed podium slabs the author has investigated in which deflection-related distress was evident in the wood-framed superstructure (primarily cracking in gypsum wallboard and racking of door and window frames). For a 13-inch (330 mm) thick normalweight non-prestressed podium slab with a maximum bay size of 29 ft x 29 ft (8.8 x 8.8 m), carrying 4 stories of wood framing above, superimposed dead load (SDL)=128 psf (6.12 kN/m^2), live load (LL)=84 psf (4.02 kN/m^2), f'_c=3,000 psi (21 MPa), f_y=60,000 psi (414 MPa), the calculated maximum computed instantaneous (elastic) deflections due to total dead and live loads in an exterior panel are:

$$\Delta_{DL} = 1.10 \text{ in (28 mm)}$$

$$\Delta_{LL} = 0.32 \text{ in (8 mm)}$$

These deflections were calculated using **PTData**, a commonly used, commercially available computer program for the design and analysis of post-tensioned and non-prestressed slabs and beams. They are the sum of the maximum panel deflections in each orthogonal equivalent frame. Using the long-term deflection coefficients specified in 9.5.2.5, the predicted deflection in this panel after 5 years (conservatively assuming no sustained live load) is:

$$\Delta_{5\,yrs} = 1.10 + 2(1.10) + 0.32 = 3.63 \text{ in (92 mm)}$$

The above calculation uses a creep factor of 2.0 in accordance with 9.5.2.5. It is possible that actual creep factors are substantially higher, which would increase the predicted long term deflection. In this type of building, construction of the wood framing begins almost immediately after the podium slab is cast and the installation of the gypsum wallboard (the last step in the framing) is generally completed within three months. Code deflection criteria are based upon the portion of the deflection that occurs after the attachment of non-structural elements likely to be damaged by large deflections. This involves considerable judgment on the part of the engineer since the non-structural elements are attached gradually over a period of time, rather than instantaneously at one point in time, and construction materials (lumber, gypsum wallboard, appliances, etc.)

are often stacked on the podium slab shortly after it is cast. The podium slab is also loaded with small forklifts and other construction equipment operating on its surface. However reasonable assumptions and estimates can be made for computing the deflections that are likely to produce damage to the non-structural elements.

If it is conservatively assumed that the damaging deflections do not start until the wallboard installation is fully complete (realistically the damaging deflections initiate when the wallboard installation begins, rather than ends), and that the entire superstructure dead load is applied instantaneously at exactly 3 months (it is actually applied gradually over the entire 3-month time period, increasing the average time for damaging deflections), the slab deflection at 3 months can be estimated as follows (using a slab concrete weight of 163 psf (7.8 kN/m^2) and a total dead load of 163+128 =291 psf (13.9 kN/m^2)):

$$\Delta_{3\,mo} = 163/291(1.10)(2)+128/291(1.10) = 1.73 \text{ in (44 mm)}$$

The portion of the deflection that occurs after connection of the framing is therefore at least 3.63 – 1.73 = 1.90 in (48 mm).

If the slab span ℓ used to determine the allowable computed deflection in accordance with Table 9.5(b) is taken as the diagonal panel dimension (see discussion below), the allowable computed deflection is 1.414x29x12/480 = 0.83

in (21 mm), less than half of the deflection which is computed to occur after connection of the framing. Clearly, this slab significantly violates the criteria of Table 9.5(b), nonetheless it is in conformance with the code since the slab thickness exceeds the minimum thickness specified in Table 9.5(c) for exterior panels of 29x12/30 = 11.6 in (295 mm).

SOME PROBLEMS WITH THE DEFINITION OF"SPAN" IN TWO-WAY SLABS

Table 9.5(b) is referenced in Section 9.5.3.4, a section that applies only to two-way construction. However the deflection limits in Table 9.5(b) are expressed in terms of a span ℓ which is defined as the *"span length of beam or one-way slab…"* Obviously the appropriate span length for two-way slabs should be stated clearly in Table 9.5(b) since the table applies to two-way slabs, not just to beams and one-way slabs.

A further desirable clarification to Table 9.5(b) would be to specify the appropriate span length for two-way slabs with rectangular panel dimensions. Since the maximum panel deflection is an additive function of the spans in each orthogonal direction, the author feels it is appropriate to base the limiting span for deflections on the panel diagonal dimension, which, of course, is a function of both spans. This eliminates concerns about whether it is appropriate to use the shorter or longer span for deflection limitations in rectangular panels. This deflection span criteria has, in fact, been used regularly in the past by experienced designers of two-way slabs and it has resulted in appropriate behavior. However this is not addressed in the current code and it should be. An appropriate new definition in Chapter 9 for *ℓ*, which would satisfy the author's concerns, is:

> *ℓ = span length of beam or one-way slab, as defined in 8.7; <u>diagonal dimension between column centerlines in two-way slab panels;</u> clear projection of cantilever, in.*

It should also be made clear that, in using the diagonal panel dimension divided by the appropriate factor as a deflection limit, the correct deflection for coMParison is the maximum deflection at the center of the panel. When equivalent frame methods are used to compute deflections, this is the sum of the computed deflections in each orthogonal frame direction. This could be clarified and stated as a footnote to Table 9.5(b).

CONCLUSIONS AND RECOMMENDATIONS

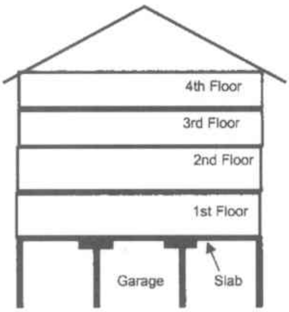
Current code deflection criteria require the computation of deflections only to justify slab thicknesses less than those specified as minimums in Table 9.5(c). Slabs with thicknesses equal to or greater than those specified in Table 9.5(c) are in conformance with the code, regardless of slab loading, and even if computations demonstrate non-compliance with Table 9.5(b). In other words, minimums establish slab thickness in all cases except where calculations justify a thinner slab.

Considering the issues addressed in this paper, a better solution would be to limit the applicability of Table 9.5(c) to slabs with superimposed loading not to exceed certain clearly stated values. These limiting values would be those that have consistently resulted in good performance. A reasonable upper limit on the superimposed loading would be the weight of the slab itself. Alternatively it would also be reasonable to limit the superimposed dead and live loads to some numeric value such as 50 psf (2.4 kN/m^2). If superimposed loads exceed the specified limits, deflection calculations and conformance to Table 9.5(b) would be required.

However the best solution, in the author's opinion, would be to require deflection computations and conformance to Table 9.5(b) in ***all*** cases, and to eliminate or modify the minimum thickness criteria in Table 9.5(c). If this were done, computations would govern slab thickness in all cases except where minimums (if they are actually necessary) would not permit a thinner slab. This is effectively the reverse of current code criteria, and in the author's opinion, much more rational. With the abundance of competent commercially available computer programs in use today for slab design, each containing sophisticated deflection algorithms that require no extra effort on the part of the designer, deflection calculations present no particular design burden. Minimum thickness criteria, independent of loading and concrete strength, should notpermit the use of thinner slabs than required by deflection calculations that are a function of ***all things*** known to affect deflection.

REFERENCE

1) *"PCA Notes on ACI 318-02 Building Code Requirements for Structural Concrete"*, Portland Cement Association, Skokie, Illinois, 2002



		DL (psf)	LL (psf)	*LL_{red} (psf)
R	ROOF	20	20	20
4th	PART.	15		
	FLOOR	25	40	24
3rd	PART.	15		
	FLOOR	25	40	24
2nd	PART.	15		
	FLOOR	25	40	24
1st	PART.	15		
	SLAB	125	40	40
	MISC.	10		
	TOTAL	290	180	132

Figure 1 – Typical Podium Slab Loading (1 psf=0.048 kN/m^2)