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Deflection Limits for Tilt-Up Wall Serviceability

The history behind changes to Chapter 14 of ACI 318

BY JOHN LAWSON

Tilt-up construction is one of the most ubiquitous forms of commercial construction in many parts of the country. Thanks to its efficient wall and roof systems, more than 700 million ft^2 (65 million m^2) of tilt-up buildings were constructed in the U.S. in 2006.[1]

Tilt-up's current popularity is even more remarkable considering that it wasn't until the 1980s that building codes began to recognize the unique design of slender tilt-up wall panels. In fact, it wasn't until ACI 318-99[2] was published that the design of slender wall panels was codified on a national level. Of course, the building code is a living document that evolves to reflect current understanding. This article provides background information on proposed revisions to ACI 318 slender wall provisions that govern the design of tilt-up wall panels.

HISTORICAL DEVELOPMENT

Prior to the development of slender wall code provisions, concrete wall thickness was controlled by limiting the height/thickness ratio. It was believed that very slender walls could buckle prematurely or deflect excessively. In the 1960s and 1970s, the ACI 318 height/thickness ratio limit of 25 for bearing walls created much thicker walls than those typically seen today. For example, a common 20 ft (6 m) high bearing wall was limited to a minimum thickness of 10 in. (240 mm).

Engineers began experimenting with new analysis techniques that included second-order effects, or $P\Delta$ moments, to avoid the height/thickness ratio limits prescribed by ACI. Many engineers used a moment magnification method in ACI 318-71[3] to account for these second-order effects, but this method was not applicable to flexural members with only a central layer of reinforcement. Even though very thin wall panels were being erected successfully in the 1970s, there was growing concern over the engineering fundamentals used to design these walls.

In response to the explosive growth of tilt-up construction being based on potentially misapplied code provisions, the Structural Engineers Association of Southern California (SEAOSC) published "Recommended Tilt-Up Wall Design,"[4] also known as the Yellow Book, in 1979. This landmark publication provided detailed design examples to appropriately consider second-order effects in slender concrete walls. It was the Yellow Book's recommendation to increase the height/thickness ratio limits if the proper second-order analysis was used.

This was a huge step forward in Southern California where the Yellow Book was quickly embraced. With height/thickness ratio limits of 36 and 42 for unstiffened and stiffened bearing walls, respectively, unstiffened 7 in. (175 mm) thick walls and stiffened 6 in. (150 mm) thick walls could now reach 21 ft (6.3 m) tall.

The Yellow Book was quickly followed by the Green Book, titled "Test Report on Slender Walls,"[5] in 1982. Based on the work of SEAOSC and the Southern California Chapter of ACI (SCCACI), this important publication contained the results of 30 full-scale slender wall tests under out-of-plane loading. Twelve of the wall specimens were tilt-up concrete walls ranging from 4-3/4 to 9-1/2 in. (120 to 240 mm) thick, representing height/thickness ratio of about 30, 40, 50, and 60. These test specimens also included eccentric gravity load from roof ledgers.

The tests were a dramatic success showing that, despite the high height/thickness ratios, the wall panels were quite capable of undergoing severe deflections while continuing to resist increasing lateral loads before yielding. One specimen didn't yield until deflecting 13 in. (330 mm), and reached its ultimate capacity after deflecting over 19 in. (480 mm). The Green Book states, "These tests proved that thin walls of this construction can handle all specified code loadings for vertical and lateral forces with reserve deflection capacities far in excess of service requirements."[5]

The very large deflections, however, raised serviceability concerns with the SEAOSC/SCCACI task committee. Slender walls designed to meet strength requirements alone and free of height/thickness ratio limits could be overly flexible, possibly resulting in permanent deformation. On several of the full-scale test specimens, rebound studies were conducted. It was found that some permanent deformation was possible in wall panels prior to reaching theoretical yield. Quoting the Green Book, "The tests demonstrated that there was no validity for fixed height-to-thickness limits, but they did reveal the need for deflection limits to control potential residual deflection in panels after service loads experience."[5] Based on their limited rebound study and much discussion, the SEAOSC/SCCACI task committee proposed a deflection limit of 1/100 the height of the panel.

As shown in Fig. 1, the full-scale tests produced a load-deflection curve that was nearly bilinear with an abrupt slope change at the cracking moment M_{cr}. Based on the test data, SEAOSC/SCCACI developed service level deflection equations for the bilinear curve. These equations became the basis for the slender wall provisions first incorporated into the 1987 Supplement to the Uniform Building Code (UBC).[6]

In the subsequent 1988 UBC,[7] service level deflection Δ_s was obtained using what was essentially an interpolation along the bilinear curve

$$\Delta_s = \frac{5M_s l_c^2}{48E_c I_g} \quad \text{for } M_s < M_{cr}$$

$$\Delta_s = \Delta_{cr} + \left(\frac{M_s - M_{cr}}{M_n - M_{cr}}\right)(\Delta_n - \Delta_{cr}) \quad \text{for } M_s > M_{cr}$$

where

M_{cr} = cracking moment based on a modulus of rupture of $5\sqrt{f'_c}$ for in.-lb units ($0.42\sqrt{f'_c}$ for SI units);

M_n = nominal moment strength at section;

M_s = the maximum moment in the wall resulting from the application of the unfactored load combinations;

E_c = modulus of elasticity of concrete;

E_s = modulus of elasticity of steel;

l_c = vertical distance between supports;

I_g = moment of inertia of the gross concrete section;

$I_{cr} = nA_{se}(d-c)^2 + \frac{bc^3}{3}$;

$A_{se} = \frac{P_u + A_s f_y}{f_y}$;

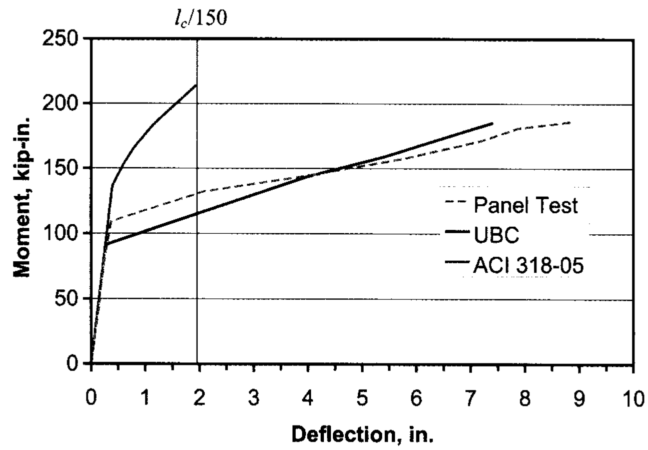


Fig. 1: Comparison of moment-deflection curve from panel test with predicted deflection based on ACI 318-05 equations and UBC 97 equations

n = E_s/E_c;

d = distance from extreme compression fiber to centroid of tension reinforcement;

c = distance from extreme compression fiber to neutral axis;

b = width of compression face of member;

P_u = factored axial load at midheight of wall, including tributary wall weight;

A_s = area of tension reinforcement;

f_y = specified yield strength of steel;

f'_c = specified compressive strength of concrete;

$$\Delta_n = \frac{5M_n l_c^2}{48E_c I_{cr}}$$

$$\Delta_{cr} = \frac{5M_{cr} l_c^2}{48E_c I_g}$$

One important difference between the Green Book and the UBC was that the deflection limit at service loads was decreased to 1/150 of the height of the panel in the UBC. This was set by consensus opinion during the 1984 to 1986 UBC code development process, but it's not clear what the basis of the decision was.

Another important aspect of both the Green Book and UBC equations was defining M_{cr} based on a modulus of rupture $f_r = 5\sqrt{f'_c}$ ($f_r = 0.42\sqrt{f'_c}$). This is only 2/3 of the traditional ACI 318 value of $f_r = 7.5\sqrt{f'_c}$ ($f_r = 0.62\sqrt{f'_c}$), but it matched the empirical test data far better. As shown in Fig. 1, uniquely defining M_{cr} and applying the UBC bilinear curve equation produces a load-deflection curve that matches the test results well. These equations continue to be included in the 1997 UBC[8] and will be used in California through the end of 2007.

INCORPORATION INTO ACI 318

With the push to develop a uniform national building code in the late 1990s, the UBC slender wall provisions were incorporated into ACI 318-99 to eliminate conflict with the 2000 IBC. Whereas the equations for determining the design moment remained essentially the same, the service level deflection equations were significantly altered. These equations remain in ACI 318-05,[9] Section 14.8.4, and are given as

$$\Delta_s = \frac{5Ml_c^2}{48E_cI_e}$$

$$M = \frac{M_{sa}}{1-\dfrac{5P_sl_c^2}{48E_cI_e}}$$

where

M = maximum unfactored moment due to service loads, including $P\Delta$ effects;

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1-\left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \leq I_g$$

= effective moment of inertia for computation of deflection (also known as Branson's equation);

$$M_{cr} = Sf_r = S\left(7.5\sqrt{f_c'}\right) \qquad \text{(in.-lb units, } \sqrt{f_c'} \text{ in psi)}$$

$$M_{cr} = Sf_r = S\left(0.62\sqrt{f_c'}\right) \qquad \text{(SI units, } \sqrt{f_c'} \text{ in MPa)}$$

S = section modulus of the gross concrete section;

M_{sa} = maximum unfactored applied moment due to service loads, not including $P\Delta$ effects; and

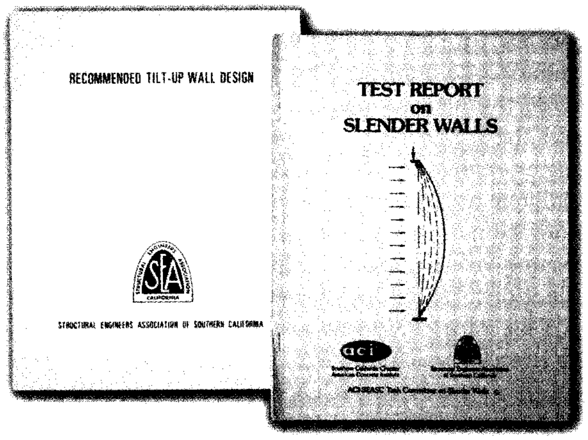
P_s = unfactored axial load at the design (midheight) section including effects of self-weight.

The most significant difference was ACI's use of Branson's equation for I_e to account for the effect of cracking on the moment of inertia instead of using the UBC bilinear load-deflection equation to determine the deflection. In addition, the value for M_{cr} used in Branson's equation was set at the traditional ACI value.

There was concern within SEAOSC that the fundamental equations developed as a result of their landmark testing program in the early 1980s had been significantly revised by ACI. In addition, the ACI commentary continued to reference this experimental research partially as the basis for these new equations. In response, SEAOSC formed a Slender Wall Task Group in 2005 to conduct a comprehensive review of the original 1981 test data and determine the validity of the UBC and ACI approaches.

WHY DOESN'T TEST DATA CORRELATE WITH ACI 318?

A fundamental question still remains: Why don't these slender walls behave in accordance with the long-standing ACI deflection equations? Neither the Yellow Book, the Green Book, nor the SEAOSC Slender Wall Task Group report discuss any theory behind the lower cracking moment M_{cr} or the bilinear moment-deflection equation. Possible answers lie in research conducted in Australia and Canada.



"Recommended Tilt-Up Wall Design," the Yellow Book, was published in 1979, followed by "Test Report on Slender Walls," the Green Book, in 1982

Australian research[10] has identified internal concrete shrinkage stresses as a significant factor affecting M_{cr} based on flat slab deflection test data. Normally, beam specimens used to test modulus of rupture are unreinforced and have little restraint, allowing free shrinkage. Once reinforcement is added, shrinkage is partially restrained as the reinforcement goes into compression, causing tensile stresses to develop in the concrete. These tensile stresses cause reinforced members to crack earlier than expected.

The following equation for M_{cr} that predicts a reduced surface stress at the initiation of cracking was adopted in 2000 by the Australian Standard for Concrete Structures AS 3600.[11] In addition to shrinkage, the Australian Code's equation for M_{cr} also includes provision for axial load stresses applied to the concrete member

$$M_{cr} = S(7.5\sqrt{f_c'} - f_{cs} + P/A) - Pe \qquad \text{(in.-lb units, } \sqrt{f_c'} \text{ in psi)}$$

$$M_{cr} = S(0.6\sqrt{f_c'} - f_{cs} + P/A) - Pe \qquad \text{(SI units, } \sqrt{f_c'} \text{ in MPa)}$$

where

$$f_{cs} = \left(\frac{1.5\rho}{1+50\rho}\right)E_s\varepsilon_{sh}$$

ρ = A_s/bd

ε_{sh} = final shrinkage strain of the concrete.

The term P/A accounts for the benefit of compression stresses or the detriment of tensile stresses on M_{cr}. In addition, any induced stresses from the eccentricity e of the axial force P are also considered. This makes the AS 3600 equation far more comprehensive than M_{cr} found using the modulus of rupture alone, which is especially important for lightly reinforced or centrally reinforced members. Recent research[12] has concluded that 2/3 of the traditional ACI 318 value for M_{cr} is simpler to use and appropriate for computing deflections, in lieu of the Australian Code method. This value for M_{cr} matches the 1997 UBC that uses $f_r = 5\sqrt{f'_c}$ ($f_r = 0.42\sqrt{f'_c}$).

Once the concrete in members with a central layer of reinforcement or light reinforcement cracks, there is usually an abrupt decrease in stiffness. Without consideration of the shrinkage effects, stiffness can be significantly overestimated, resulting in an underprediction of deflections. Using the moment-deflection curve shown in Fig. 1 for Panel #27 as an example, the M_{cr} produced during the test was about 9.1 ft-kips (12.3 kN·m). The M_{cr} predicted by the 1997 UBC, AS 3600, and ACI 318-05 equations are 7.6, 8.9, and 11.4 ft-kips (10.3, 12.1, and 15.5 kN·m), respectively, with the AS 3600 equation producing the closest estimate of M_{cr}.

Research[13] has also identified significant limitations with Branson's equation for I_e when applied to thin concrete members with a central layer of steel. Branson's equation, first published in 1965, was based on larger test beams with a ratio of gross/cracked moment of inertia (I_g/I_{cr}) set at 2.2. When this ratio exceeds a value of about 3, ($I_g/I_{cr} > 3$), the use of Branson's equation leads to poor predictions of deflection. Slender concrete walls are far above this limit, with common ratios ranging from 15 to 25 for single layer reinforced walls and 6 to 12 for double layer reinforced walls; thus deflection is underpredicted. The main culprit for this underprediction is the lack of proper consideration of tension stiffening in Branson's equation. Recommendations to replace Branson's equation with a more accurate equation incorporating tension stiffening effects have recently been proposed.[13,14]

PROPOSED REVISIONS IN ACI 318-08

The public comment period for ACI 318-08 concluded at the middle of last month. Unless there are changes based on these comments, slender wall design provisions in ACI 318-08 will more closely match the original recommendations of the SEAOSC/SCCACI task committee that were incorporated into the UBC. The changes were brought about, in part, by a SEAOSC Slender Wall Task Group that re-evaluated the test results published in the Green Book and conducted a study comparing ACI 318-02[15] and the 1997 UBC slender wall design provisions. The Task Group reported their findings in their "UBC 97 and ACI 318-02 Code Comparison Summary Report"[16] published in 2005.

They concluded that the 1997 UBC equations match the test data well, but the ACI 318-02 equations (that were unchanged in ACI 318-05) did not correlate well with the test data and typically underestimate service load deflections. Figure 1 provides a typical comparison of the UBC and ACI 318-05 equations with the original test data for one slender wall specimen. The other wall specimens had similar comparisons.

The revised deflection equations are

$$\Delta_s = \left(\frac{M_a}{M_{cr}}\right)\Delta_{cr} \text{ for } M_a \le (2/3)M_{cr}$$

$$\Delta_s = (2/3)\Delta_{cr} + \left(\frac{M_a - (2/3)M_{cr}}{M_n - (2/3)M_{cr}}\right)\left(\Delta_n - (2/3)\Delta_{cr}\right) \text{ for } M_a > (2/3)M_{cr}$$

where

$$\Delta_{cr} = \frac{5M_{cr}l_c^2}{48E_cI_g}$$

$$\Delta_n = \frac{5M_nl_c^2}{48E_cI_{cr}}$$

It should be noted that, although the equations for Δ_{cr} and Δ_n are identical to the equations from the 1988 UBC, M_{cr} in the equation for Δ_{cr} is still based on the higher modulus of rupture for concrete traditionally used in ACI 318, but this is rectified by the use of the 2/3 factor in the calculations for Δ_s. The ACI 318-08 equations also eliminate the use of Branson's equation in favor of the bilinear equation used in the UBC when calculating deflections for slender wall panels.

The new equations produce a moment-deflection curve that is identical to the UBC prediction and more closely matches the data. As Tables 1 and 2 illustrate, the new equations provide conservative results when compared with data from the 12 tilt-up wall panel tests.

Further comparing the test data in Table 1, the equation for M_{cr} currently in ACI 318-05 overestimates the wall's cracking moment by 26% on average. Because of the drastic change in the bilinear load-deflection curve at M_{cr}, this overestimation results in a significantly lower calculated panel deflection. In contrast, the UBC and proposed ACI 318-08 revisions conservatively underestimate M_{cr} by 16% on average.

Table 2 compares the load-deflection accuracy of the two methods with the 12 tilt-up wall panel tests. The moments are tabulated for a deflection of 1/150 of the height of the panel. The inaccuracies of M_{cr} and Branson's I_e for these thin panels combine to cause the ACI 318-05 results to significantly overestimate the corresponding moments. The ACI 318-05 approach overestimated the moments by 77% on average. By comparison, the UBC and proposed ACI 318-08 revisions consistently provided a close approximation that was 13% lower, on average, than the observed moment at a deflection of 1/150 of the height of the panel.

FURTHER RESEARCH NEEDED

Clearly, slender wall design has come a long way since the early days of height/thickness ratios. The ability to design

TABLE 1:
COMPARISON OF OBSERVED AND CALCULATED CRACKING MOMENTS

Panel No.*	Thickness, in. (mm)	M_{cr}† observed, ft-kips (kN·m)	M_{cr}‡ UBC, ft-kips (kN·m)	M_{cr}‡ ACI 318-05, ft-kips (kN·m)
19	9.6 (244)	21.9 (29.7)	19.5 (26.4)	29.2 (39.6)
20	9.4 (239)	22.3 (30.2)	18.7 (25.4)	28.0 (38)
21	9.5 (241)	21.8 (29.6)	19.1 (25.9)	28.6 (38.8)
22	7.4 (188)	12.8 (17.4)	11.6 (15.7)	17.3 (23.5)
23	7.3 (185)	12.9 (17.5)	11.4 (15.5)	17.1 (23.2)
24	7.4 (188)	15.0 (20.3)	11.5 (15.6)	17.2 (23.3)
25	6.1 (155)	10.4 (14.1)	7.9 (10.7)	11.9 (16.1)
26	5.9 (150)	10.3 (14.0)	7.3 (9.9)	11.0 (14.9)
27	6.0 (152)	9.1 (12.3)	7.6 (10.3)	11.4 (15.5)
28	4.8 (122)	6.8 (9.2)	4.9 (6.6)	7.4 (10.0)
29	4.8 (122)	5.2 (7.1)	4.8 (6.5)	7.2 (9.8)
30	4.9 (124)	5.2 (7.1)	5.1 (6.9)	7.6 (10.3)

*Panel numbers correspond to full scale testing program by SEAOSC/SCCACI.[5] All panels are 24 ft (7.3 m) tall, 4 ft (1.2 m) wide, and reinforced with four No. 4 (No. 13) bars.
†Cracking moment estimated from load-deflection test data.
‡Cracking moment calculated using actual section and material properties measured for each specimen.[5]

TABLE 2:
COMPARISON OF OBSERVED AND CALCULATED MOMENTS AT A LATERAL DEFLECTION OF 1/150 OF THE HEIGHT OF THE PANEL

Panel No.*	$M_{L/150}$† observed, ft-kips (kN·m)	$M_{L/150}$‡ UBC, ft-kips (kN·m)	UBC difference, %	$M_{L/150}$‡ ACI 318-05, ft-kips (kN·m)	ACI difference, %
19	23.3 (31.6)	20.6 (27.9)	−12	50.8 (68.9)	118
20	23.5 (31.9)	20.1 (27.3)	−14	48.7 (66.0)	107
21	24.1 (32.7)	20.3 (27.5)	−16	49.7 (67.4)	106
22	14.6 (19.8)	13.9 (18.8)	−5	28.7 (38.9)	97
23	14.7 (19.9)	12.3 (16.7)	−16	27.6 (37.4)	88
24	17.4 (23.6)	15.2 (20.6)	−13	28.9 (39.2)	66
25	12.8 (17.4)	10.5 (14.2)	−18	18.9 (25.6)	48
26	11.9 (16.1)	9.9 (13.4)	−17	17.2 (23.3)	45
27	10.8 (14.6)	9.4 (12.7)	−13	17.8 (24.1)	65
28	7.3 (9.9)	6.0 (8.1)	−18	10.8 (14.6)	48
29	6.9 (9.4)	6.2 (8.4)	−10	10.7 (14.5)	55
30	6.3 (8.5)	6.1 (8.3)	−3	11.1 (15.0)	76
Average	—	—	−13	—	77

*Panel numbers correspond to full scale testing program by SEAOSC/SCCACI.[5] All panels are 24 ft (7.3 m) tall, 4 ft (1.2 m) wide, and reinforced with four No. 4 (No. 13) bars.
†Moment at $\Delta = 1/150$ of the height of the panel from load-deflection test data.
‡Moment at $\Delta = 1/150$ of the height of the panel calculated using actual section and material properties measured for each specimen.[5]

efficient concrete wall systems with greater predictability has helped the industry, but there are issues that remain unresolved and additional research is needed.

One area that needs clarification is determining what constitutes service load levels for deflections under seismic loads. This issue will be discussed in an upcoming article.

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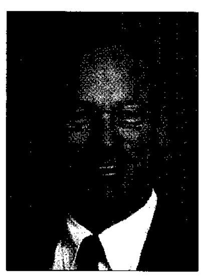

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