# Concrete Slender Wall Design – *Back to the Future*

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## Abstract

Slender concrete walls incorporated into tilt-up construction over the past 60 years have performed remarkably well under out-of-plane wind and seismic loads. While issues associated with seismic wall anchorage gave this form of construction a black-eye in the early days, the concrete walls themselves have always performed very well even as far back as their first use in the early 1900s. Yet the height-to-thickness limitations in 1985 and earlier model building codes were proved irrational and were removed. We as engineers expect building codes to evolve and advance the state of the art with each successive edition. The latest edition of ACI 318-08 has significantly revised the slender walls design procedures, yet not necessarily advancing the state of the art. The latest ACI 318 edition is largely revising slender wall design back to match equations found in the old 1997 UBC.

This paper revisits the historical effort SEAOSC played in the development of the original slender wall provisions in the late 1970s, and why ACI is now revising their slender wall design provisions to agree with concepts developed over 30 years ago by SEAOSC. While these original concepts were based on empirical data from full-scale tests conducted in the early 1980s, only within the last five years have we really begun to fully understand the behavior of these thin concrete members when subjected to combined axial load and large horizontal forces.

## Historical Background

Tilt-up panels were developed in the pre-WWI era as an erection technique to facilitate construction of large concrete wall panels without forming both sides in-place. In 1909, Col. Robert Aiken, described an innovative method of casting panels on tilting tables and then lifting them into place by means of specially designed mechanical jacks [Spears, 1980]. During the ensuing years, this technique was used for constructing target abutments, barracks, ammunition and gun houses, a mess hall, low cost housing, factory buildings and churches [Leabu, 1980.]

After WWII, from the late 1940s to the 1960s, reinforced concrete tilt-up walls became a great innovation and construction advancement. Walls could be built, not with expensive wall-forms but cast on the concrete slabs on the ground and lifted into final position serving as architectural enclosures or elements. Developer clients on industrial and commercial buildings, particularly supermarkets and large warehouses, demanded taller walls than the maximum 16 feet 8 inch allowed for an 8-inch thick concrete wall, based on a height-to-thickness limit of 25. The first report "Technical Bulletin Number 2," on tilt-up wall construction, covering the design and construction practice of tilt-up at that time, was published in 1949 by the Structural Engineers Association of Southern California.

## Height-to-Thickness Ratio

Prior to 1985, building codes requirement for the design of concrete wall panels were based on arbitrary height-to-thickness limitations. A minimum wall thickness of six inches was required. A height-to-thickness ratio limitation of 25 was imposed on bearing walls, and 30 for non-bearing walls. Such height-to-thickness ratios increased to 36 when second-order effects were accounted for in the wall panel design in accordance with ACI 318 Chapter 10's requirements for slender compression members.

The increased use of tilt-up concrete walls for commercial and industrial buildings led to trends toward designing slender walls using pseudo second-order analyses. At that time, a second order analysis was permitted by the building code, and that led to a trend towards designing slender walls with little or no concern for stiffness, thus possibly affecting long-term serviceability. In 1974, Portland Cement Association developed design aids for load bearing walls [Kripanarayanan, 1980]. The design aids were compatible with ACI 318-77 Section 10.10.1 for the design of compression members using a second-order frame analysis. The design aids helped engineers in designing panels under eccentric axial loads combined with horizontal forces.

This PCA method also included analysis and design for wall panels supported on isolated footings. The design utilized the deep beam provisions of the code in addition to applying a reduction factor to account for out-of-plane buckling. In the 1960s and 1970s, many tilt-up structures were designed and built using cast-in-place pilasters and founded on isolated footings. Design of tilt-up panels often approached a height-to-thickness ratio of 48.

### City of Los Angeles Restrictions on Slender Walls

The explosive growth of tilt-up, without an embraced engineering approach, became a major concern among building officials as well as some practitioners. Many engineers agreed that the height-to-thickness ratios in the building code were conservative. In order to side step the height-to-thickness limits, engineers began using alternative approaches such as moment magnification to account for the second-order effects of slender wall panels. Various jurisdictions, including the City and County of Los Angeles Building and Safety, began imposing policies regarding the arbitrary upper limit on height-to-thickness restrictions. But such design deviations from the bulding code were permitted only on a job to job basis.

In 1977, the SEAOSC Board appointed an Ad Hoc Committee to review the issues on tilt-up wall construction and to give recommendations on appropriate design practice. The Ad Hoc Committee was chaired by Robert White with fourteen members representing the private and public sectors and industry. While the Committee was handicapped by the lack of tests on slender wall panels, the committee did come to conclusion that the height-to-thickness ratio could be increased beyond the traditional code requirements of 25, provided P-Δ (second-order) effects would be considered in design. In 1979, a report was published on "Recommended Tilt-up Wall Design," also known as the "Yellow Book" [SEAOSC, 1979]; which served as the basis of alternate design for slender concrete tilt-up panels.

The Yellow Book essentially helped to achieve uniformity in enforcement in Southern California area. A brief summary of the "Yellow Book" method is given below:

- $\ell_c/h \leq 36$ for panels supported top & bottom
- $\ell_c/h \leq 42$ for panels supported all four edges
- $P/\Delta$ effect accounted for in design
- Design based on mid-height deflection under the nominal moment strength, $M_n$

$$M_n = A_{s(eff)} f_y (d-a/2)$$

*Where*: $A_{s(eff)} = (A_s + P_u/f_y)$
$a = (A_s f_y + P_u) \div (0.85 f_c' b)$
$\Phi = 0.90 - 2P_u/f_c' A_s \geq 0.7$

$$M_u = w_u l_c^2/8 + P_{u1}e/2 + (P_{u}1+P_{u2}/2)\Delta_n$$

*Where*: $\Delta_n = 5M_n l_c^2 \div (48 E_c I_{cr})$
$E_c = 33 w^{1.5}\sqrt{f_c'}$
$I_{cr} = nA_{s(eff)}(b-c)^2 + bc^3/3$

When $M_u \leq \Phi M_n$, then the wall section is adequate for slenderness requirements.

The underlying philosophy of this methodology was predicated on an idealized moment deflection curve as shown in Figure 1. The deflection at the nominal moment strength was considered as the controlling deflection for the P-Δ effect meeting the intent of ACI 318 section 10.10.1. The Ad Hoc Committee further recommended that physical tests be conducted to substantiate this design technique.

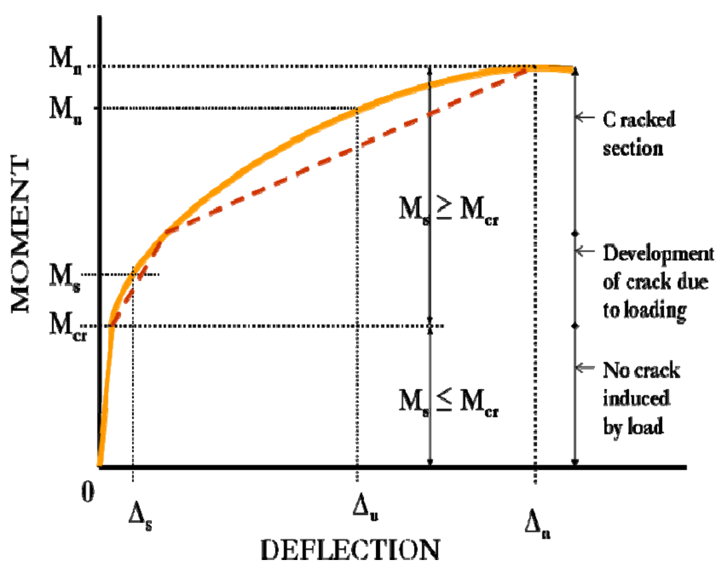


**Fig. 1 – Idealized Moment Deflection Curve**

### Full Scale Test of Thin Wall Panels

In late 1979, the Southern California Chapter of the American Concrete Institute under the direction of Technical Committee Chair Joseph Dobrowolski approached the

SEAOSC Board to organize a Task Committee on Slender Walls, and the SEAOC Board soon agreed to such joint effort. James S. Lai, who was active in the Board of Directors for both organizations, was named as liaison member for the Task Committee. The Committee was active between 1979 and 1982. Members of the Committee were appointed to include a balance representation from the Structural Engineers Association of Southern California and Southern California Chapter of the American Concrete Institute. Jim Armhein of Masonry Institute of America was keenly interested to promote tests on masonry panels as well and became a strong supporter of the test program. Professor Larry Selna of UCLA was at that time active in ACI Committee 441 and played a strong role on the academia aspect of the test program. The Slender Wall Task Committee was chaired by William M. Simpson* SE, Ralph S. McLean* SE (project director), and supported by Samy A. Adham CE, James E. Amrhein SE, John Coil SE, Joseph A. Dobrowolski* CE, Ulrich A. Foth* SE, James R. Johnson* SE, James S. Lai SE, Donald E. Lee SE, Lawrence G. Selna SE, and Robert E. Tobin* CE. [Note * indicates members are deceased.] The planning and conducting of the wall panel tests would not have been successful without the cooperation of many engineering offices, building officials and the construction industry [ACI-SEAOSC, 1982].

**Fig. 2 –Slender Wall Test Task Committee (1980)**

The goal was to test full scale thin concrete and masonry wall panels that exceeded the code limitations of height-to-thickness ratios. In organizing the testing program, the Task Committee obtained a testing site in a concrete subcontractor's yard in Irwindale, California. A slab-on-grade was cast on which twelve concrete panels were cast, cured and stored waiting to be lifted. The panels were subjected to combine eccentric vertical and lateral loads to simulate gravity loads and wind or earthquake lateral force. Pre-test steps included fabrication of a test loading frame and an air bag, securing equipment for instrumentation and planning for the loading sequence.

All panels were 24 feet 8 inches in height by 4 feet in width. All panels were reinforcement with 4- #4 vertical bars as tabulated in Table 1.

**Table 1 – Concrete Panel Data**

| Thickness (inches) | h/t Ratio | Reinforcing Ratio ρ (%) |
|---|---|---|
| 9.50 | 30 | 0.18 |
| 7.25 | 40 | 0.46 |
| 5.75 | 50 | 0.58 |
| 4.75 | 60 | 0.70 |

It was interesting to note that there was a definite two-part load deflection performance as shown in Figure 3. The walls behaved elastically until approximately two-thirds of the traditional modulus of rupture was reached ($5\sqrt{f_c'}$) and the initial crack formed. As the lateral load was increased, additional flexural cracking occurred, and the deflection rapidly increased. Figure 3 shows the load-deflection characteristic of four of the test panels. The deflection and load was increased until failure or an extreme deflection was reached. Results of the full-scale tests showed that there was no lateral instability from the combined lateral and eccentric vertical loading as shown in Figure 4.

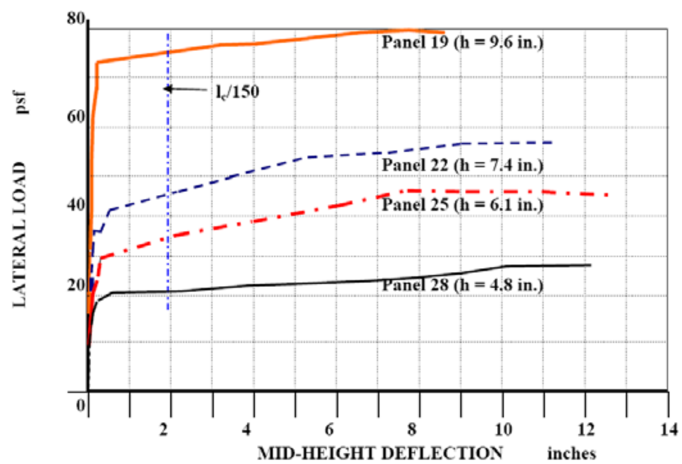


**Fig. 3 – Test Panel Load Deflection Characteristic**

**Fig. 4 – Test Panel Showing no Instability**

Following the successfully completion of the tests, the Task Committee worked on the resolutions, distillation and codifying of the data and writing of a report that became better known as the “Green Book” [ACI-SEAOSC, 1982]. The Committee concluded that design of slender wall panels required not only adequate strength and safety to resist vertical and lateral loads but also a new concept to address stiffness concerns.

This concept was wall serviceability after undergoing the code specified lateral force. The slender walls had to be serviceable and not experience damage or permanent deformation under service level forces. This brought in the limitation of wall deflection. The amount of deflection was initially stated as 0.01 times the height of the wall (L/100). During the review by the ICBO code officials and structural engineers the deflection was reduced to 0.007 times the height of the wall (L/150) out of concerns for incompatible deflection of other non-structural elements. The deflection limitation was the first time that serviceability was even considered and written into the building code for wall panels [Amrhein, 2007].

The alternate slender wall design procedure for slender concrete wall panels was introduced in the 1987 Supplement of Uniform Building Code (UBC). The design method incorporated the combined load effects due to eccentric axial loads and the P-Δ effect. Strength requirements were considered when selecting the amounts of reinforcement. Deflection under service load was established to give a reasonable limitation on the stiffness of the wall panels. Due to the diligent work of the committee, the restrictive regulations on height and thickness were changed and new design parameters introduced into the code to allow safe and serviceable tall slender walls under vertical and lateral wind or seismic loads. The slender wall design provisions in the UBC continued under this philosophy with little change from its introduction in 1987 until the 1997 UBC.

**Fig. 5 – Test Panel Showing Cracking Pattern**

## National Acceptance

In the late 1990s with the push to develop a uniform national building code, the UBC slender wall provisions were incorporated into ACI 318-99. Of the other two regional codes, the BOCA and SBC, no other competing provisions existed setting the stage for a smooth transition of the slender wall design philosophy.

However, whereas the equations for determining the design moment remained essentially the same, the service level deflection equations were significantly altered by ACI during this transition to ACI 318. These revised equations remain in ACI 318-05, Section 14.8.4 and are given as:

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

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

*where:*

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

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

$I_e =$ Effective moment of inertia for computation of deflection (also known as Branson's Equation);

$M_{cr} = Sf_r = S\left(7.5\sqrt{f_c'}\right)$; moment at initial cracking

$S =$ Section modulus of the gross concrete section;

$M_{sa} =$ Maximum 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.

When comparing the UBC approach with the new ACI approach, the most significant difference was ACI's use of Branson's equation for $I_e$ to account for the moment of inertia's reduction due to cracking. The previous UBC approach and SEAOSC philosophy used a bilinear load-deflection equation to determine the deflection. Another significant change was the value for $M_{cr}$ used in Branson's equation was set at the traditional ACI value of $7.5\sqrt{f_c'}$ instead of SEAOSC's recommended $5\sqrt{f_c'}$.

## SEAOSC Concerns

Within SEAOSC there was concern that the fundamental equations developed from their full-scale testing program had been significantly altered by ACI 318. In addition, the ACI 318 commentary continued to reference SEAOSC's 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 current UBC and new ACI approaches.

The SEAOSC Task Group found that the UBC methodology matched well with the full-scale test data collected in the 1980s. However, the Task Group found that the ACI methodology was a poor match for the observed stiffness of the full-scale test data. More specifically, the new ACI 318 equations significantly underestimated the onset of cracking $f_r$ and $M_{cr}$ and significantly underestimated the panel's stiffness after cracking $\Delta_s$. Figure 6 dramatically depicts the large disparity between the two approaches.

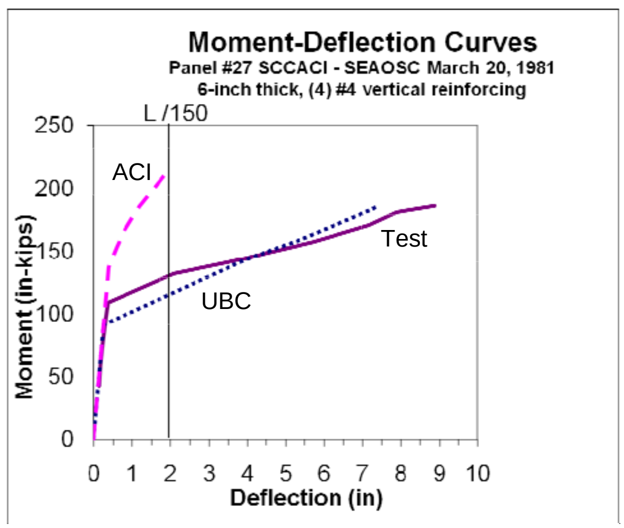


**Fig. 6 – ACI and UBC Comparison to Test Data**

The Task Group issued their opinions in a report [SEAOSC 2006] and recommended that original SEAOSC methodology, which was incorporated into the UBC, be codified again at the national level. The two authors of this paper worked towards ICC or ACI adoption of the past UBC methodology based on their Task Group findings. In 2006, the ACI 318 committee was very receptive the Task Group findings and incorporated the necessary changes into the ACI 318-08 edition.

## ACI 318-08 Provisions

The slender wall provisions of ACI 318-08 no longer contain Branson's formula for computing the effective moment of inertia, and has substituted in its place a bilinear equation similar to the UBC approach.

$$\Delta_s = 0.67\Delta_{cr} + \left(\frac{M_s - 0.67M_{cr}}{M_n - 0.67M_{cr}}\right)(\Delta_n - 0.67\Delta_{cr}),$$

One standout difference is ACI's use of $0.67\Delta_{cr}$ and $0.67M_{cr}$ instead of the UBC's $\Delta_{cr}$ and $M_{cr}$. $\Delta_{cr}$ and $M_{cr}$ in the ACI equation for $\Delta_s$ are still based on the higher modulus of rupture $f_r$ for concrete traditionally used in ACI 318. The 0.67 factor is simply ACI's approach to rectifying the disparity between UBC's $f_r = 5\sqrt{f_c'}$ based on test data and ACI's $f_r = 7.5\sqrt{f_c'}$ customary equation. Instead of revising ACI's modulus of rupture equation to reflect the test data of initial cracking, ACI took the approach to simply ratio the affected attributes $\Delta_{cr}$ and $M_{cr}$ (5/7.5 = 0.67).

The new equations produce a moment-deflection curve that is nearly identical to the UBC results and closely matches the test data. As Tables 1 and 2 illustrate, the new equations provide conservative results when compared with data from the twelve tilt-up wall panel tests in the 1980s. This contrasts sharply to the non-conservative results of ACI 318-05 and before.

**Table 1 – $M_{cr}$ Comparisons**

| Panel No.(1) | Thickness (in) | $M_{cr}$(2) observed (ft-kips) | $M_{cr}$(3) UBC (ft-kips) | $M_{cr}$(3) ACI 318-05 (ft-kips) |
|---|---|---|---|---|
| 19 | 9.6 | 21.9 | 19.5 | 29.2 |
| 20 | 9.4 | 22.3 | 18.7 | 28.0 |
| 21 | 9.5 | 21.8 | 19.1 | 28.6 |
| 22 | 7.4 | 12.8 | 11.6 | 17.3 |
| 23 | 7.3 | 12.9 | 11.4 | 17.1 |
| 24 | 7.4 | 15.0 | 11.5 | 17.2 |
| 25 | 6.1 | 10.4 | 7.9 | 11.9 |
| 26 | 5.9 | 10.3 | 7.3 | 11.0 |
| 27 | 6.0 | 9.1 | 7.6 | 11.4 |
| 28 | 4.8 | 6.8 | 4.9 | 7.4 |
| 29 | 4.8 | 5.2 | 4.8 | 7.2 |
| 30 | 4.9 | 5.2 | 5.1 | 7.6 |

(1) Panel numbers correspond to full-scale testing program by SEAOSC/SCCACI. All panels are 24-feet tall, 4-feet wide and reinforced with four #4 rebar.
(2) Cracking moment estimated from Load-Deflection test data.
(3) Cracking moment calculated using actual section and material properties measured for each specimen

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 significant error in 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 twelve tilt-up wall panel tests. The acting moments are tabulated for a resulting deflection of 1/150 of the height of the panel. The inaccuracies of $M_{cr}$ and Branson's $I_e$ combine to cause the ACI 318-05 results to significantly overestimate of corresponding moments. The ACI 318-05 approach overestimated the acting moments by 77% on average. By comparison, the UBC and proposed ACI 318-08 revisions consistently provided a close, conservative moment approximation, within 13% on average.

## Mechanics of Actual Slender Wall Behavior

The comparisons depicted in Tables 1 and 2 make clear something has gone astray when applying fundamental ACI equations to these slender concrete walls. Neither the SEAOSC Yellow Book, the Green Book, nor the SEAOSC Slender Wall Task Group report discuss any theories behind the lower cracking moment $M_{cr}$ or the empirically derived bilinear moment-deflection equation. Possible answers lie in research conducted in the United States, Australia and Canada.

Australian research [Gilbert, 1999] built upon the work of Andrew Scanlon and confirmed internal concrete shrinkage stresses as a significant factor affecting $M_{cr}$ based on flat slab deflection test data. Normally, beam specimens used to determine modulus of rupture $f_r$ are unreinforced and have little internal 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 internal 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 AS3600 [Gilbert, 2001]. In addition to shrinkage, the Australian Code's equation for $M_{cr}$ also includes a provision for axial load stresses applied to the concrete member.

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

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

where:

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

$\rho = A_s/bd$

$\varepsilon_{sh}$ = final shrinkage strain of the concrete.

The term *P/A* accounts for the benefit of compression stresses or the detriment of tensile stresses on influencing the cracking moment $M_{cr}$. Also, any induced tensile stresses from an eccentric axial load *P* are considered. This makes the AS3600 equation far more comprehensive, which is especially important for lightly reinforced or centrally reinforced members. Recent research though has concluded that the use of 2/3 $M_{cr}$ is simplier and quite appropriate for computing deflections, in lieu of the Australian Code method [Scanlon, 2008].

This value for $M_{cr}$ matches the 1997 UBC, which uses:

$$f_r = 5\sqrt{f'_c} \text{ (psi) or}$$

At the onset of cracking, members with a central layer of reinforcement (or lightly reinforced) will have an abrupt decrease in stiffness. Because the internal reinforcement lowers the cracking moment $M_{cr}$ due to shrinkage, ignoring this $M_{cr}$ reduction will significantly overestimate the member's stiffness and thus under predict the deflections. As an example, Panel #27 of the full-scale testing program was analyzed using AS3600. The AS3600 equation for $M_{cr}$ predicts a cracking moment of 8.9 ft-kips compared with 9.1 ft-kips observed during the tests. As can be seen in Table 1, the AS3600 equation produces the closest estimate of $M_{cr}$ for this test specimen compared with the 1997 UBC and ACI 318-05 approaches.

Research [Bischoff, 2007] has also identified significant limitations with Branson's equation for $I_e$ when applied to thin concrete members with a central layer of steel.

**Table 2 – $M_{L/150}$ Comparisons**

| Panel No.(1) | $M_{L/150}$(2) observed (ft-kips) | $M_{L/150}$(3) UBC (ft-kips) | UBC error % | $M_{L/150}$(3) ACI 318-05 (ft-kips) | ACI error % |
|---|---|---|---|---|---|
| 19 | 23.3 | 20.6 | -12% | 50.8 | 118% |
| 20 | 23.5 | 20.1 | -14% | 48.7 | 107% |
| 21 | 24.1 | 20.3 | -16% | 49.7 | 106% |
| 22 | 14.6 | 13.9 | -5% | 28.7 | 97% |
| 23 | 14.7 | 12.3 | -16% | 27.6 | 88% |
| 24 | 17.4 | 15.2 | -13% | 28.9 | 66% |
| 25 | 12.8 | 10.5 | -18% | 18.9 | 48% |
| 26 | 11.9 | 9.9 | -17% | 17.2 | 45% |
| 27 | 10.8 | 9.4 | -13% | 17.8 | 65% |
| 28 | 7.3 | 6.0 | -18% | 10.8 | 48% |
| 29 | 6.9 | 6.2 | -10% | 10.7 | 55% |
| 30 | 6.3 | 6.1 | -3% | 11.1 | 76% |
| | | Average = | -13% | Average = | 77% |

(1) Panel numbers correspond to full-scale testing program by SEAOSC/SCCACI. All panels are 24-feet tall, 4-feet wide and reinforced with four #4 rebar.
(2) Acting Moment at Δ=L/150 estimated from Load-Deflection test data.
(3) Acting Moment at Δ=L/150 calculated using actual section and material properties measured for each specimen.

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 three ($I_g/I_{cr} > 3$), the use of Branson's equation leads to poor predictions of deflection. Slender concrete walls are far above this limit, with common $I_g/I_{cr}$ ratios ranging from 15 to 25 for single-layer reinforced walls and 6 to 12 for double-layer reinforced walls; thus deflection is significantly under predicted. The main culprit for this under prediction is the lack of proper consideration for tension stiffening in Branson's Equation. Recommendations to replace Branson's equation with a more accurate equation incorporating tension stiffening effects similar to the Eurocode have been proposed recently [Bischoff, 2007; Gilbert, 2007; Lawson, 2007].

### Service Level Loadings

Thus far, this paper has been focusing on our ability to accurately predict the slender wall behavior, especially deflections under service level loads. While we may be getting more accurate in computing the response of these panels, there still is a great deal of uncertainty as to what service loads actually are.

Historically, service level loads were simply unfactored allowable stress loadings. Under the older Uniform Building Code, wind and seismic lateral loads were computed at an allowable stress level and factored up for strength based design. With the transition in the profession heading towards strength based design across all material groups, seismic loadings are now computed at the strength level and must be factored downward for allowable stress design. Currently, both wind and seismic load combinations involve load factors to adjust to allowable stress levels and presumably service level loadings, thus service level loads are no longer "unfactored" loads.

It is helpful to discuss at this point the intent of service level loading checks. With the increasing awareness of performance based design concepts, the intention of service level checks are to ensure a higher level of performance under lower, but more frequent, levels of earthquake or wind forces. In slender wall design, sufficient panel stiffness is considered important to prevent permanent deformations under smaller earthquakes or winds that may occur frequently.

Interestingly, ASCE 7-05 contains Appendix C which is a helpful beginning to understanding service level loadings. Appendix C explains the intent of service level loadings is to address frequent events that have a 5% probability of being exceeded annually. Appendix C's wind load combination is given as:

$$D + 0.5L + 0.7W$$

Compared to past allowable stress load combinations, this provides a lower design criteria, but no longer based on an arbitrary methodology without probability. This same $0.7W$ factored wind load can also be found in 2006 IBC Table 1604.3, footnote f, for wall design. Note: Appendix C was omitted in the first printing of ASCE 7-05 but became available as errata.

Unfortunately, ASCE 7-05 does not provide a discussion on developing a similar load combination for seismic design. Trying to develop a simple load combination for seismic with the intent of a 5% annual probability of exceedence is not possible due to the different approaches taken for risk exposure across the United States. The design spectral accelerations incorporated into the building code are not based on a uniform probability, but instead have been modified for different regions of the United States. The eastern part of the country is largely based on a probability methodology while the western coast is primarily based on a deterministic methodology. Here in California, the deterministic approach prevails and is not associated with how frequent specific ground motions occur, but instead how large an earthquake can a specific fault generate.

This lack of uniformity between east and west regions of the United States, and the lack of a uniform probability approach in California for ground motions, results in the inability to apply a simple one-size-fits-all load factor for service loads. Subsequently, ACI 318-08's commentary Section R14.8.4 for alternate slender wall design recommends simply applying the following load combination for service level seismic loadings:

$$D + 0.5L + 0.7E$$

This load combination is realistically a step back in time to our old allowable stress force levels which traditionally have been used without a problem. It should be pointed out that in low seismic regions of the United States, the $0.7E$ greatly overestimates the expected force levels associated with 5% annual probability, and there may be some merit in the criticism that this force level is too conservative in areas of low to moderate seismic risk. This is an area that could benefit from further research.

### Developing New Computer Models

As has been demonstrated in this paper, sometimes the best intentions to provide state-of-the-art engineering concepts to concrete member design fail to capture the actual behavior. Our expertise in developing complex mathematical models to predict actual behavior is no substitute for actual full-scale tests when possible.

Modeling the non-linear properties of concrete in finite element programs has long been a difficult problem. As these programs become more pervasive in the engineering office, it is important for practitioners to understand the underlying assumptions made. Simply relying upon our theoretical understanding of concrete behavior and extrapolating it to slender wall members is potentially risky without fully appreciating the lessons learned from the Slender Wall Task Committee's work in the early 1980s. Developers of today's computer engineering programs should use the Slender Wall Task Committee's test data for additional validation of their work.

## Conclusion

Building codes continue to evolve as new knowledge is gained from science and experience. The hope is that we further the state-of-the-art and provide safer, more efficient, buildings with each code cycle. Occasionally, we get ahead of ourselves or lose the necessary perspective, and must reevaluate what has transpired in the building code. Despite our first instincts, don't always dismiss it when someone says, *"I've been doing it this way for twenty years…."*

This paper is dedicated to those members of the Slender Wall Task Committee who have passed on, but left this legacy for others to benefit from.

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