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Revisions to Tilt-Up Concrete Building Design: The backstory behind changes encompassed in 2009 International Building Code

John Lawson SE
California Polytechnic State University, San Luis Obispo

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Abstract

The 2009 International Building Code (IBC) has in its scope some significant revisions that greatly affect the design of tilt-up concrete buildings, but to the lay engineer it appears that some of these provisions are a step backwards. With the IBC's referencing standards including ACI 318-08 (concrete) and the 2008 NDS SDPWS (timber), these documents have significant changes that become effective with the adoption of the IBC. We as engineers expect building codes to advance the state of the art with each successive edition. The 2009 *International Building Code* (IBC) and the *Building Code Requirements for Structural Concrete* (ACI 318-08) have significant revisions affecting the design of tilt-up buildings, but in some cases not necessarily advancing the state of the art. For example, the latest ACI 318 edition is largely revising slender wall design back to match equations found in the old 1997 UBC. In addition, the latest IBC edition is carving out an exception to an especially troublesome code provision due to an oversight in the adoption process. Other changes in timber design are mostly administrative or incorporating errata from the last code cycle.

This paper provides the background for these changes and reviews their impact to tilt-up concrete building design and wall panel design. In addition, a historical context is provided where if may provide some clarity.

Revisions to the cracked moment of inertia (I_{cr})

Since the original incorporation of slender wall provision into the *Uniform Building Code* (UBC), the actual steel areas were allowed to effectively increase in the calculations to reflect the impact of vertical compression load. This "effective steel area" reflected the prestressing effects of compression load on the section by converting the compression force into and effective steel reinforcing area for flexural design. The following equation illustrates the approach currently in ACI 318-05:

$$I_{cr} = \frac{E_s}{E_c}\left(A_s + \frac{P_u}{f_y}\right)(d-c)^2 + \frac{l_w c^2}{3}$$

This equation's approach is to take the applied vertical compression load in the wall, assumed at the wall's center line, and increase the steel area artificially through the term P_u/f_y. Because the steel's location was assumed to be typically at the wall's center line, this is a rational approach.

The cracked moment of inertia I_{cr} is useful in computing a slender concrete wall's ultimate moment including secondary effects through the following ACI 318-08 equation.

$$M_u = \frac{M_{ua}}{1 - \dfrac{5P_u l_c^2}{(0.75)48E_c I_{cr}}}$$

Unfortunately, this approach is not rational when the reinforcing steel is not aligned with the resultant compressive force location. This situation most frequently occurs in walls with two curtains of reinforcing steel. This error has now been corrected with a new term added to the cracked moment of inertia equation provided in the ACI 318-08 Section 14.8 code provisions. Using a new term *h/2d* in ACI equation 14-7, the effective steel area is made rational again for either single or double curtain reinforcing conditions.

$$I_{cr} = \frac{E_s}{E_c}\left(A_s + \frac{P_u}{f_y}\frac{h}{2d}\right)(d-c)^2 + \frac{l_w c^2}{3}$$

The impact of this code revision will be that flexural design of double curtain walls will be slightly more restrictive.

Minimum Reinforcing Around Wall Openings

A long standing code requirement has been that concrete wall openings have a minimum of two #5 reinforcing bars around their perimeter. For single curtain walls, this provision has been revised to a minimum of one #5 reinforcing bar around the openings in ACI 318-08 Section 14.3.7.

In addition, the minimum extension of these bars past the edge of the openings has traditionally been 24-inches, an arbitrary length. In a more rational approach, this code requirement has been lengthened to the development of F_y in the bar.

The impact of this is likely minimal in California or other seismically active states where reinforcing is typically much larger than the minimum vertically.

Slender Wall Serviceability Provisions

The most significant code changes affecting tilt-up wall design are the changes to the slender wall design in ACI 318-08 Section 14.8. This code section contains the design provisions for slender wall design loaded out-of-plane. To protect slender walls from experiencing permanent deformation under frequent service-level loads, a deflection equation and limitation is provided. In order to better appreciate the changes in the ACI 318-08 provision, a brief historical discussion is provided.

In late 1979, the Southern California Chapter of the American Concrete Institute in conjunction with the Structural Engineers Association of Southern California embarked on a landmark testing program of concrete and masonry slender walls.

The main goal was to test full scale slender concrete and masonry walls that exceeded the code limitations at the time in terms of height-to-thickness ratios. Both the concrete and masonry wall panels were subjected to a combine eccentric vertical load and lateral loads to simulate gravity loads and wind or earthquake lateral forces. An air bag loading apparatus with instrumentation was utilized to load twelve tilt-up wall panels across their four-foot width and 24-foot height. There were four different wall thicknesses, and all were reinforced with four #4 vertical reinforcing bars. The various characteristics of the tested panels are 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

As visible in Figure 1, the load-deflection curve was essentially bilinear for all panels. 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 1 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.

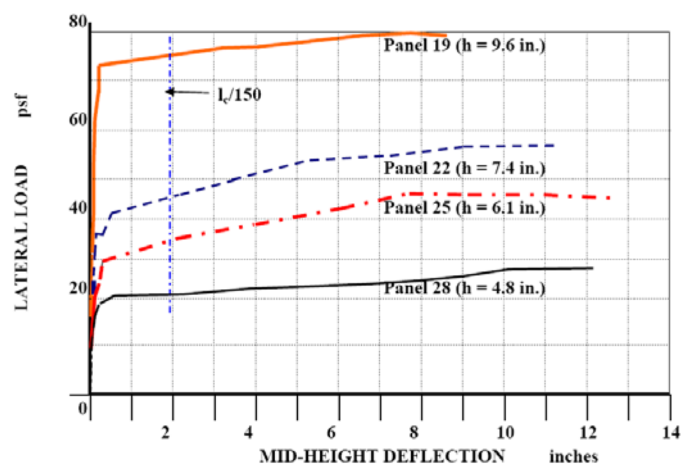

Fig. 1 – Test Panel Load Deflection Characteristic

After the testing program was completed, the Task Committee worked on 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. The serviceability provisions in the current code are a direct outgrowth of this testing program.

A rebound study was conducted during the testing program and it was determined that L/100 was an appropriate limit to service level deflections. This limitation was later revised to L/150 under the UBC when it was incorporated in the 1987 supplement to the UBC. The deflection limitation was the first time that serviceability was even considered and written into the building code for wall panels [Amrhein, 2007].

The UBC's new slender wall 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. The UBC slender wall provisions continued under this philosophy with little change from its introduction in 1987 until the 1997 UBC.

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:

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

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

where:

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

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

$I_e =$ 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 $5\sqrt{f_c'}$.

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 Δ_s. Figure 2 dramatically depicts the large disparity between the two approaches.

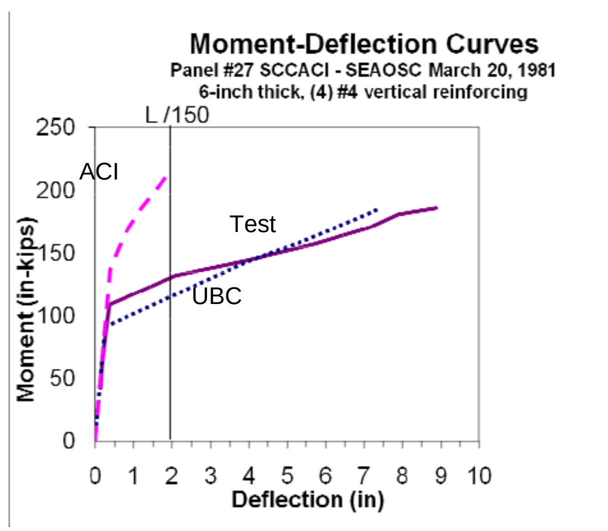

Fig. 2 – ACI and UBC Comparison to Test Data

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

Table 2 – M_{cr} Comparisons

(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

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.

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

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

One standout difference is ACI's use of $0.67\Delta_{cr}$ and $0.67M_{cr}$ instead of the UBC's Δ_{cr} and M_{cr}. Δ_{cr} and M_{cr} in the ACI equation for Δ_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 Δ_{cr} and M_{cr} (5/7.5 = 0.67).

The new equations produce moment-deflection curves that are nearly identical to the UBC results and closely match 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.

Further comparing the test data in Table 2, 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 3 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.

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.

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%

Table 3 – $M_{L/150}$ Comparisons

(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.

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.

$$M_{cr} = S(7.5\sqrt{f'_c} - f_{cs} + P/A) - Pe$$

(in.-lb units, $\sqrt{f'_c}$ in psi)

$$M_{cr} = S(0.6\sqrt{f'_c} - f_{cs} + P/A) - Pe$$

(SI units, $\sqrt{f'_c}$ in MPa)

where:

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

$\rho = A_s/bd$

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 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 simpler 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 2, 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.

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 Loads – What are they?

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.

Ductility Provisions for Wall Anchorage

In Seismic Design Category C and higher, ACI 318-05 Appendix D requires anchorage to concrete to be designed to behave in a ductile manner. The engineer has another option allowed by the 2006 IBC to design the connection for 2.5 times more force in lieu of satisfying ductility. As will be now discussed, this approach unfortunately failed to recognize the intent of the wall anchorage forces incorporated into the 2006 IBC. The 2006 IBC wall anchor forces are essentially the same as those found in the previous 1997 UBC.

First, a little background is necessary. Following the 1994 Northridge earthquake, surveys of damage to concrete and masonry buildings with flexible roof diaphragms revealed that very limited amounts of wall anchorage ductility were present to resist the induced forces. Brittle tensile failures in steel wall anchorage straps were especially troublesome. In addition, boundary nailing in plywood diaphragms tore out of the plywood edges due to wall anchorage elongation. New code provisions were introduced into the 1997 UBC to address the nonductile wall anchorage behavior observed in the Northridge earthquake.
1997 UBC Section 1633.2.8.1 was chiefly written to address many of the wall anchorage issues spotlighted in the Northridge earthquake. The lack of observed ductility and the need for greater anchorage strength were the reasons behind 1997 UBC Section 1633.2.8.1 Items 1 and 4. Wall anchorage forces to flexible diaphragms in Seismic Zones 3 & 4 were increased 50% (a_p=1.5) and steel elements had an additional 1.4 force multiplier.

The intent of Section 1633.2.8.1 (items 1, 4, and 5) was for the wall anchorage system to resist brittle failure when subjected to maximum expected roof accelerations. Based on observations of Northridge earthquake damage, it was deemed best to resist brittle failure through the use of significantly higher design forces in conjunction with anticipated material overstrength instead of any reliance on ductility. As a result, material-specific load factors were introduced to provide a uniform level of protection against brittle failure (1.4 steel, 0.85 wood, 1.0 concrete/masonry). This approach is well documented in the 1999 SEAOC Recommended Lateral Force Requirements and Commentary (The Blue Book) [Reference C108.2.8.1].

As further evidence of the intent of these wall anchorage provisions, the 1999 SEAOC Blue Book Commentary states that the reduced R_p value for nonductile and shallow anchorage does not apply to wall anchorage designed using this overstrength approach of Section 1633.2.8.1 [Reference C108.2.8.1].

In the development of ASCE 7-05, the intent was to maintain the same wall anchorage equation between the 1997 UBC and ASCE 7-05 for flexible diaphragms in high seismic zones. The wall anchorage provisions of ASCE 7-05 Section 12.11.2.1 are directly incorporated from the 1997 UBC Section 1633.2.8.1. Substituting $C_a = 0.4S_{DS}$ (2003 NEHRP Commentary), it can be confirmed that Eq. 12.11-1 is generally equivalent to the 1997 UBC.

Through an unrelated parallel effort, ACI 318-05 Appendix D Sections D.3.3.4 and D.3.3.5 require anchorage ductility in moderate and high seismic zones. ACI's ductility requirement conflicts with the intent behind Section 12.11.2.1 at wall anchorage situations. Furthermore, 2006 IBC Section 1908.1.16 allows an additional 2.5 load factor on top of ASCE forces in lieu of the ACI ductility requirement. This stacking of load factors on top of load factors and ductility requirements is in conflict with the original intent of the wall anchorage provisions.

To summarize, the 1997 UBC and subsequent ASCE 7-05 implement very high wall anchorage force levels to achieve uniform protection against brittle failure without reliance upon ductility. This was achieved using a rational approach considering inherent overstrength. Through the incorporation of ACI 318 Appendix D, anchorage ductility requirements were inadvertently added to these special wall anchorage situations in conflict with the original intent of the provisions. Furthermore, the 2006 IBC force multiplier of 2.5 is redundant to the original force increase behind the UBC and ASCE wall anchorage provisions.

Achieving anchorage ductility under ACI 318 Appendix D is very difficult for tilt-up construction with flexible diaphragms. For the ductility condition to be met, steel anchor strength must be weaker than the concrete breakout strength. Because tilt-up walls are inherently thin slender wall designs, anchor embedment depth is limited, making it difficult to increase. In several parametric studies, it is apparent that the ductility provision encourages smaller diameter steel anchors or thicker concrete walls for deeper embedments. Neither of these approaches seems beneficial.

Another unintended consequence of providing ductile anchorage is the potential elongation of the steel causing boundary nailing at plywood diaphragms to tear out of the sheathing edges under maximum seismic force levels. Similar concerns exist for edge welding along steel deck diaphragms at the wall panels.

Using the 2006 IBC 1908.1.16 alternative, the forces are increased to an extreme level due to the 2.5-times load increase previously mentioned. In several parametric studies, this results in a larger number of thin anchors rods spread out over a larger connection area. Spreading these anchor rods out likely results in non-uniform anchorage force distribution, and instead concentrates the forces over the closest few rods, potentially resulting in a progressive rod failure.

Recognizing the conflict within the building code, the International Code Council accepted a proposed IBC code revision from this author at the Final Action Hearings in Minneapolis, Minnesota in September 2008. This code revision creates an exception to the requirement for ductility or the 2.5 times overstrength provisions when the design forces are computed using wall anchorage provisions at flexible diaphragms. The following statement now is contained in the 2009 IBC Section 1908.1.9:

> *"Anchors designed to resist wall out-of-plane forces with design strengths equal to or greater than the force determined in accordance with ASCE 7 Equation 12.11-1 or 12.14-10 need not satisfy Section D.3.3.4 [nor] D.3.3.5."*

Revisions to Wood Sheathed Diaphragms

With the IBC's coordination with and incorporation of the 2008 NDS *Seismic Design Provisions for Wind and Seismic* (SDPWS), several changes affect tilt-up concrete buildings with wood diaphragms. When diaphragm shear forces exceed 820 lbs/ft (ASD), the traditional diaphragm shear table is insufficient and the designer must seek design solutions from the High-Load Diaphragm table in the 2009 IBC, and is now also in the 2008 SDPWS.

The 2006 IBC had incorporated the high-diaphragm concept from APA's evaluation report ICC-ES 1952, but inadvertently omitted the important nail spacing and stagger diagrams from the report. This omission left confusion among designers who had never consulted the original evaluation report that the table was based on. In the new 2009 IBC, this omission has been rectified and the diagrams are now included.

In an effort to further consolidate wood framing provisions into the NDS publications, the 2009 IBC has removed the provisions and equations for diaphragm deflections utilizing nailing, and instead is deferring to the SDPWS where the same provisions are stated. One exception is that the deflections of diaphragms utilizing staples is still in the 2009 IBC, however staples are not very commonly used, and it is the author's recommendation to focus on staples as a solution only when considering repair options in the field to minimize splitting.

Conclusion

The inevitable changes that occur between code cycles often leave engineers bewildered as to the reason behind the new provisions. In a general sense, we expect building codes 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, code provisions are created in a hurry without the necessary perspective to insure intent is honored. With this code cycle, tilt-up construction's state of the art is being advanced in some instances and being returned appropriately to the past in others.

The process of building code development involves many volunteers who pursue rational code provisions that push the state of the art forward for safer and better structures. While not infallible, those who work towards this goal should be commended.

Fig. 3 – The SCCACI / SEAOSC "Green Book" Team

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