ASCE 7-05 reference section(s)	ACI 318-05 Reference section(s)	2006 IBC reference section(s)
12.11	14.8 21.7 21.8 21.13	1908.1.16 1908.1.8 1908.1.13 2305

Introduction

The reinforced concrete tilt-up building is the most popular form of light industrial and low-rise commercial construction in the Western United States and is a significant portion of new construction nationwide. This popularity is primarily driven by the construction speed and economical nature of tilt-up construction. Architectural acceptance has become more widespread as tilt-up construction has adapted to the demands of taller buildings with better aesthetics involving irregular plan shapes, more glass, and accent treatments. Tilt-up construction is becoming more common for office buildings, assembly occupancies, and even schools. Originally a one-story form of construction, tilt-up buildings are now commonly two and three stories.

Unfortunately, the uniqueness and rapid evolution of tilt-up construction have made it a challenge for seismic provisions in building codes to keep pace. Poor performance in past earthquakes has been responsible for significant revisions to building codes and in some jurisdictions mandated seismic retrofit requirements of older tilt-up buildings. This article describes unique problems associated with tilt-up seismic design, how the past has shaped current recommended practice, and insights from recent research on tilt-up building behavior.

Tilt-up buildings consist of reinforced concrete wall panels that are formed, cast, and cured on the building floor slab or adjacent “waste” slab and then tilted up into a vertical position with a lifting crane. Surrounding the perimeter of the building, these concrete wall panels are typically between six and twelve inches thick, and are both gravity load-bearing and seismic force-resisting. Once the walls are in place and temporarily braced, a roof structure is erected consisting of either metal decking or panelized wood sheathing (structural-use panels) over steel or wood framing members. The most popular framing system in use today is the “hybrid” roof system consisting of a panelized wood roof of OSB (oriented strand board) and 2x4 framing, supported on factory installed wood nailers attached to the top chord of open-web steel joists (trusses). The roof structure is connected to the interior wall face, allowing the walls to extend above the roof as a parapet. See Figure 1.

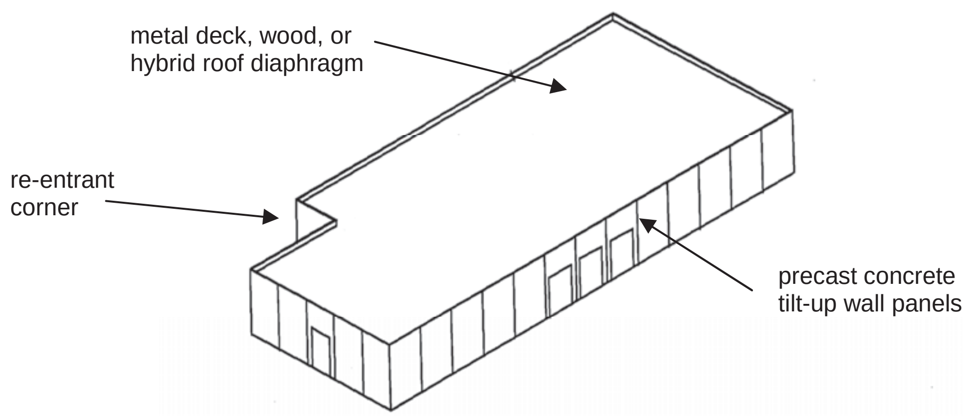

Figure 1. Typical structural features of a tilt-up building

The most critical component of tilt-up building seismic performance is the anchorage of the wall to the roof structure. This anchorage is accomplished with embedded concrete anchors or straps attached to the roof framing system. Finally, the wall panels are connected to the slab-on-grade through lapped reinforcing within a pour strip or fill-in strip of concrete floor. It is also strongly recommended that the wall panels be connected to the foundations in high seismic zones to provide a positive load path.

Historical Background

Many seismic code provisions that have evolved over the years envision either classic frame structures with rigid diaphragms or light frame buildings with small lightweight diaphragms. Tilt-up construction is unique with its many different material combinations and has unique performance issues that have not generally entered the code until evidence of a problem from a failure. Tilt-up construction has experienced many earthquake-induced failures in the past (EERI 1988, SEAONC 2001). The tilt-up provisions that do exist are scattered throughout the code, primarily because tilt-up does not fit neatly into one material type. IBC Chapter 16 (Structural Design), Chapter 19 (Concrete), Chapter 22 (Steel), and Chapter 23 (Wood) all contain important provisions for the design of the various materials associated with diaphragms, collectors, and wall anchorage, and with the main lateral force-resisting system.

Even though the code has become more prescriptive, there are many aspects of design that vary significantly between engineers. There still is no clear direction provided on some important engineering issues such as the following:

- How to properly tie into a re-entrant wall;
- How shear loads are distributed along a line of perforated shear walls;
- Whether wall ties should be designed for compression or tension only.

Walls Loaded Out-of-Plane. Prior to the development of slender wall code provisions in the 1980s, concrete walls were limited to arbitrary height/thickness (h/t) ratios creating walls much thicker than typically seen today. It was believed that very slender walls could buckle prematurely or deflect so much as to not be serviceable after earthquakes.

To justify thinner walls, panels were joined together with cast-in-place pilaster stiffeners formed between adjacent panels. The concrete wall panels spanned horizontally out-of-plane between the pilasters, and the pilasters spanned vertically between the floor slab and a supporting roof beam. Horizontal concrete panel reinforcing extended into the cast-in-place joint, effectively making all the panels behave monolithically.

In the 1970s, engineers began experimenting with wall panel designs spanning vertically between the floor slab and roof, leaving the panel joints dry and caulked with waterproof sealants. Pilasters cast with the panels were still provided (usually on just one panel edge), but only for the purpose of providing roof beam gravity load reactions. The panels were now themselves spanning vertically without the reliance on the pilasters, behaving as tall, slender walls. ACI 318 limited bearing wall slenderness to a height-to-thickness (h/t) ratio of 25, and these tall, slender walls did not conform to the typical UBC or ACI code provisions for concrete walls. The slenderness restriction prevented economical use of vertically spanning tilt-up panels.

Instead, engineers began experimenting with new analysis techniques to include second order effects, or $P\Delta$ moments, as a means to circumvent the arbitrary h/t limits prescribed by ACI. Inadvertently, many engineers used a moment magnification method in ACI 318-71 to account for these second order effects, but this method was not developed for flexural members with only a central layer of steel. Another approach used was to conduct rigorous strain compatibility reviews or to use published papers such as the 1974 report, *Tilt-Up Load-Bearing Walls—A Design Aid*, by the Portland Cement Association (Kripanarayanan 1984). Even though very thin wall panels were being erected successfully in the 1970s, there was growing concern over the engineering fundamentals behind the analysis of these walls.

In response to the explosive growth of tilt-up construction that was based on potentially misapplied code provisions, the Structural Engineers Association of Southern California (SEAOSC) published in 1979 their *Recommended Tilt-*

Up Wall Design, also known as the “Yellow Book” (SEAOSC 1979), and was quickly followed in 1982 by the “Green Book” titled *Test Report on Slender Walls* (ACI-SEAOSC 1982). Based on the work of SEAOSC and the Southern California Chapter of ACI, this important publication contained the results of thirty full-scale tests of slender walls under out-of-plane loading (twelve slender concrete walls with the others being masonry), and it confirmed the recommended design provisions of the “Yellow Book” but without arbitrary height-to-thickness (h/t) limits.

These provisions recognized that with light axial loads, tilt-up wall panels behave more like flexural members than compression wall sections, thus eliminating the need for arbitrary height-to-thickness limits. Both *P*-delta effects and eccentric loading were deemed very important considerations, because slender panels are capable of undergoing large out-of-plane deflections, and the provisions provided equations that estimated the non-linear deflection characteristics out-of-plane. Also, the flexural reinforcement ratio and axial loading were restricted to ensure ductile flexural yielding while mitigating sudden buckling collapse. One of the inherent advantages of using what became known as slender wall design is the reasonably thinner wall panels, which in turn reduce the seismic force at the roof diaphragm. Even though the SEAOSC provisions were not incorporated into the UBC until 1988, engineers were quick to embrace these guidelines in their designs.

The very large deflections observed in the testing program raised serviceability concerns with the SEAOSC/SCCACI task committee. Slender walls designed to strength requirements alone, free of h/t ratios, could be overly flexible, possibly resulting in permanent deformation. On several of the full-scale test specimens, rebound studies were conducted and 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.” Based on their limited rebound study and much discussion, the SEAOSC/SCCACI task committee recommended an L/100 deflection limit.

The recommended provisions of the Yellow Book and Green Book were the basis of the first building code requirements in the 1988 UBC. The one important difference between the Green Book and the UBC was the deflection limit at service loads was more restrictively set at L/150. This was set by consensus opinion during the 1984-1986 UBC code development process, but it is not clear what rational basis there was behind the revision.

Another important aspect of both the Green Book and UBC equations was defining the concrete cracking moment, M_{cr} , based on the modulus of rupture, $f_r \quad 5\sqrt{f'_c}$, of concrete. This is two-thirds of the traditional ACI equation $f_r \quad 7.5\sqrt{f'_c}$, but it matched the full-scale test data far better. By uniquely defining M_{cr} and applying a bilinear curve equation, the UBC load-deflection curve matched well with the test results. See Figure 2. These equations continued to be incorporated up through the 1997 UBC.

Historically, the out-of-plane seismic performance of slender tilt-up wall panels has been relatively good, with failures of a panel itself very rare. Often the worst out-of-plane loading occurs during crane lifting in the construction process. In the 1987 Whittier Narrows, California earthquake, some out-of-plane damage was observed at the sides of large wall openings, but the damage was associated with post-construction saw cut openings installed without strengthening or engineering (EERI 1988).

Today, out-of-plane tilt-up wall panel design is incorporated into ACI 318-05 Section 14.8, and is still largely based on the original SEAOSC testing and recommendations.

The lateral force coefficient for out-of-plane structural wall forces for Seismic Design Category B and above is provided in ASCE 7-05 Section 12.11.1 as F_P=0.40$S_{DS}IW_w$. Using the lateral force coefficient to determine the out-of-plane wall forces, an engineer normally selects vertical design strips to determine moment and reinforcing requirements. At wall piers between wall openings and panel joints, the design strip usually is the entire pier width, with loads accounting for the increased seismic tributary loading associated with the panel portions above and below

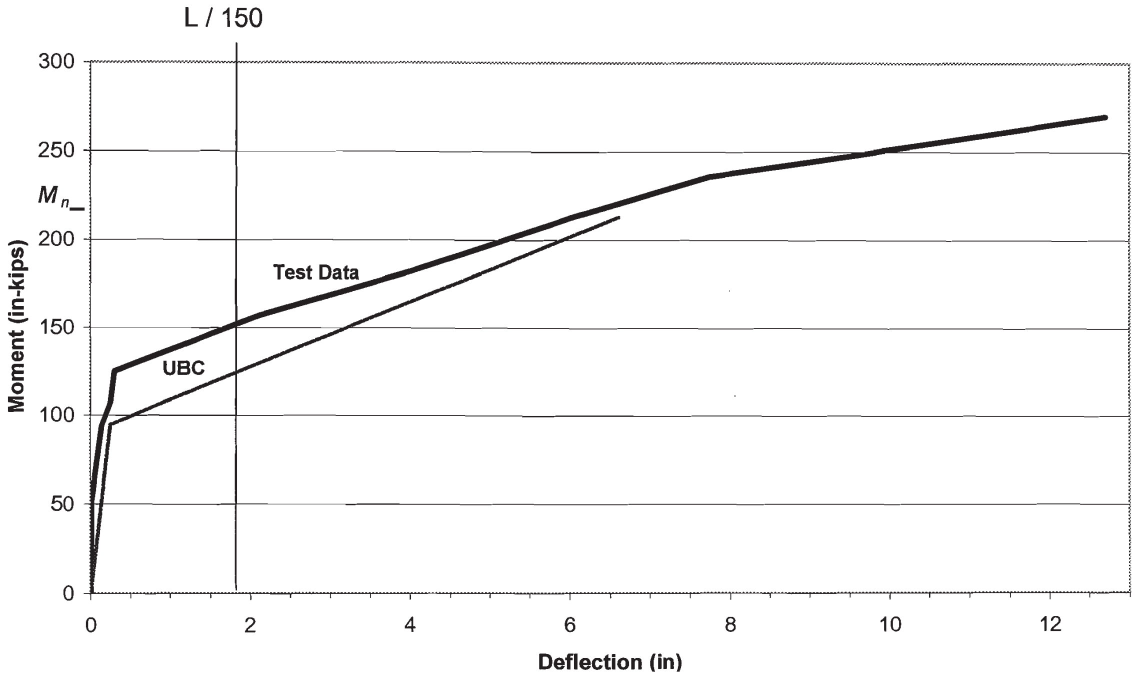

Figure 2. Moment-Deflection Curves Comparing Test Data with UBC
Panel #25 SCCACI-SEAOSC March 20, 1981; 6-in-thick reinforced slender wall

the openings. This approach typically neglects any additional strength or stiffness provided by the portions of the wall above and below the openings, using a simple strip of uniform width as an analytical tool.

Cantilever parapets that are part of a continuous wall element are checked separately for higher seismic forces per ASCE 7-05 Section 13.3.1. Unlike the structural wall panels, the parapet forces are based on nonstructural component equations. It is not appropriate to use the higher cantilever parapet forces to offset the wall positive bending moment below the roof.

Whereas the UBC equations checking strength remained essentially in concert with the ACI 318 adoption, the service level deflection equations were significantly altered. The most significant difference was use of Branson's equation for the effective moment of inertia, I_e, in ACI in place of the UBC bilinear load-deflection equation. In addition, the value for M_{cr} within Branson's equation was set at the traditional ACI value.

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

Where,

$$M_{cr} \quad Sf_r \quad S\left(7.5\sqrt{f'_c}\right)$$

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 318 Commentary continued to reference the SEAOSC 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.

The SEAOSC Slender Wall Task Group reported their findings in its *UBC 97 and ACI 318-02 Code Comparison Summary Report* (SEAOSC 2006). The Task Group concluded that the 1997 UBC equations match the test data well, yet the ACI 318-02 equations do not correlate well with the test data and typically underestimate the service load deflections. Figure 3 provides a typical comparison of the UBC and ACI equations with the original test data of one slender wall specimen. The other wall specimens had similar comparisons. The ACI equations for service level deflection fail to properly estimate the cracking moment and the bilinear nature of the load-deflection curve.

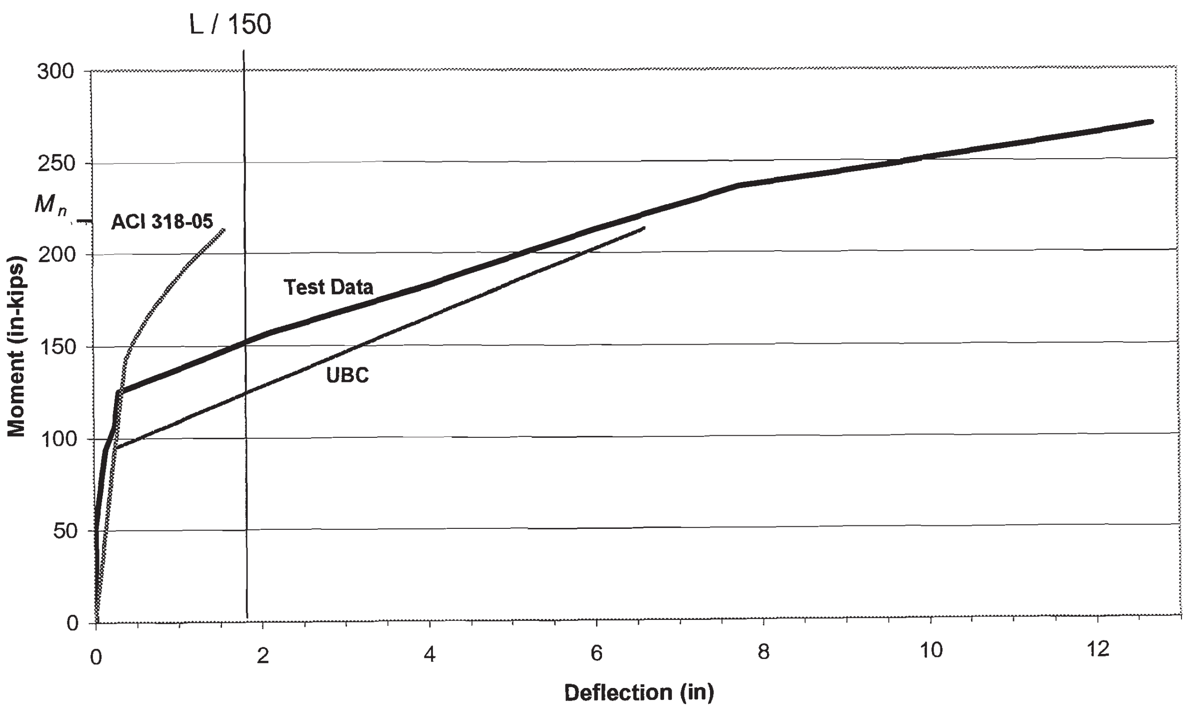

Figure 3. Moment-Deflection Curves Comparing Test Data with ACI 318-05 and UBC

The current ACI 318 slender wall provisions have been revised for ACI 318-08 resulting in service level deflection equations more in line with that of the 1997 UBC.

The reason why concrete slender walls behave so differently than predicted by traditional ACI equations has not been clear until recently. Neither the Yellow Book, the Green Book, nor the SEAOSC Slender Wall Task Group discuss any theory behind the lower cracking moment M_{cr} or the bilinear moment-deflection equation unique to slender walls. Research conducted in Australia and Canada has provided good explanations to the disparity between traditional ACI deflection theory and the full-scale slender wall tests (Lawson 2007b).

The conflict between M_{cr} observations and the ACI M_{cr} equation is associated with internal concrete shrinkage stresses first investigated in deflecting flat slabs (Scanlon and Murray 1982). Normally, beam specimens tested for modulus of rupture are unreinforced and have no restraint, allowing free shrinkage. Once reinforcing is added, shrinkage is partially restrained as the reinforcing goes into compression, causing surface tension stresses to develop

in the concrete. These pre-existing tension stresses cause reinforced members to crack earlier than expected in plain concrete. Flexural members with low reinforcement ratios such as tilt-up slender walls are especially sensitive to shrinkage restraint stress, thus significantly reducing the effective cracking moment (Scanlon and Bischoff 2007, Gilbert 2001).

The stiffness of members with only a central layer of reinforcing or that are lightly reinforced depends greatly on the tensioning effects of the plain concrete near the surface. Once this concrete cracks, there is usually an abrupt change in stiffness. Without consideration of the shrinkage effects, stiffness can be significantly underestimated and thus unexpected deflections can occur under service level loads. Other building codes around the world already have similar provisions for a reduced effective cracking moment or modulus of rupture due to shrinkage restraint, including the Australian Standard for Concrete Structures AS3600, the Canadian Code CSA A23.3 (for slabs), and the Eurocode. A comparison of these codes in conjunction with available testing data indicates that use of $0.67M_{cr}$ or use of $f_r = 5\sqrt{f'_c}$ in typical slender wall design is appropriate instead of the current ACI 318 (Scanlon and Bischoff 2007, Lawson 2007b).

The reduced effective cracking moment outlined above does not fully explain the better fit of the bilinear curve used by the UBC. Bischoff and Scanlon (2007) studied this issue with respect to concrete reinforced with Fiber Reinforced Polymer Bars and found similar issues persist among other thin lightly reinforced concrete members, including concrete tilt-up wall panels

Bischoff identified significant limitations with Branson's equation for I_e when applied to thin concrete members with a central layer of steel, and he has proposed a solution. Branson's Equation, first published in 1963, was based on larger test beams where the ratio of gross/cracked moment of inertia (I_g/I_{cr}) was typically 2.2. Bischoff found that Branson's equation became a poor predictor of deflection when this ratio exceeded three ($I_g/I_{cr} > 3$). Slender concrete walls are far above this, with common values ranging from 15 to 25 for single layer reinforced walls and 6 to 12 for double-layer reinforced walls. Thus deflection is under predicted.

Bischoff has proposed a new equation as a replacement to Branson's for effective moment of inertia. This equation matches well with both larger flexural beams and thin slender walls, effectively transitioning seamlessly to a near bilinear load-deflection curve at high I_g/I_{cr} ratios.

$$I_e = \frac{I_{cr}}{1-\eta\left(\frac{M_{cr}}{M_a}\right)^2} \leq I_g \qquad \text{where} \qquad \eta = 1-\frac{I_{cr}}{I_g}$$

Use of Bischoff's equation with a reduced $0.67M_{cr}$ provides a single approach to computing reasonably accurate deflections in lightly reinforced slender tilt-up walls (Scanlon and Bischoff 2007, Lawson 2007a). Figure 4 compares the load-deflection curve using ACI 318-05, replacing Branson's I_e with Bischoff's I_e, and replacing the ACI M_{cr} with $0.67M_{cr}$. This provides a much better fit than the current ACI 318-05 deflection equation. While this proposed curve is nearly parallel to the test data, it appears to consistently overestimate the service level deflections. This discrepancy maybe attributed to the panels being tested at 160 days instead of reaching ultimate drying shrinkage and thus a resulting lower $M_{cr.}$.

Reveals, recesses, and form-liner surface treatments must all be considered as they reduce the wall section net thickness and flexural depth of the reinforcing. Narrow, reasonably shallow reveals that only occur occasionally across the panel height often can be justified as not significantly affecting the panel stiffness for determination of out-of-plane deflections, but these still must be included in the moment strength analysis if they exist at critical design sections.

The equations in ACI 318-05 Section 14.8 are very straightforward and can easily be written in spreadsheet form for the engineer's own use. Several commercially available software packages are available on the market for designing or analyzing these walls. The widely available programs make basic assumptions associated with wall openings and are to be considered only estimates, but are appropriate practice. The true out-of-plane behavior of a tilt-up wall panel with openings is more like a two dimensional flat slab with penetrations requiring use of general-purpose finite element programs to obtain more accurate results. However, the inherent changes in concrete stiffness associated with flexural cracking due to the various seismic stresses and original lifting stresses makes the likelihood of a very accurate finite element analysis remote and therefore not warranted.

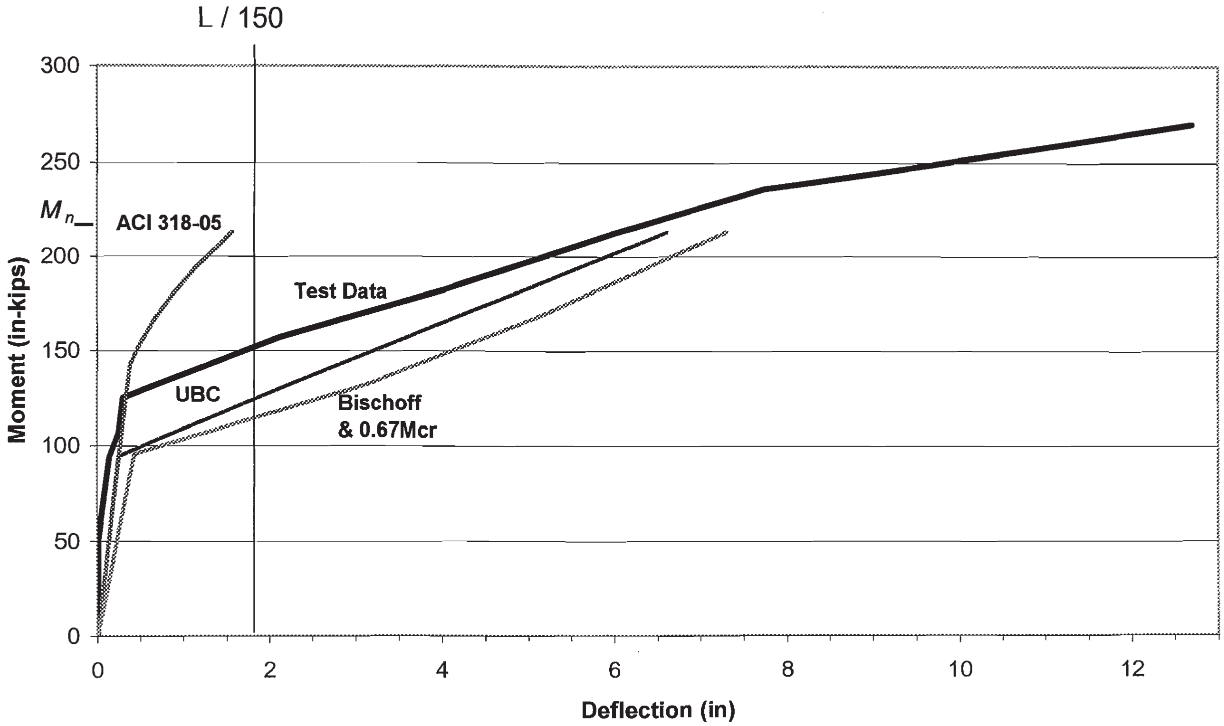

Figure 4. Moment-Deflection Curve Comparison with Modified I_e and M_{cr} Values

Engineers using finite element programs to design panels are cautioned to research the underlying assumptions and equations. Much of what is known about slender wall behavior is based on empirically derived equations from test data. Computer results from programs based entirely on theoretical formulas may be in error.

Wall Anchorage, Subdiaphragms and Continuity Ties. In the mid 1960s, engineers who worked closely with contractors began using wood ledgers as the default wall anchorage with anchor bolts clustered around wood purlins. This resulted in no positive direct tie anchoring the perimeter concrete wall panels to the supporting roof structure. Instead, the roof plywood sheathing was simply nailed a wood ledger which was bolted to the inside face of the wall panels (SEAONC 2001, chapter 3). 2x subpurlins and 4x purlins were supported from the ledger by metal hangers. The glulam beams and occasionally tapered steel girders were supported on top of the pilasters and had seat connections with minimal tie capacity. This indirect tie arrangement relied upon the wood ledger in cross-grain bending, a very weak material property of wood.

In the 1971 San Fernando Earthquake, tilt-up buildings performed poorly. Many wood ledgers split length-wise due to cross-grain bending loads, and plywood edge nailing pulled through plywood panel edges as the result of wall anchorage tension loads. Partial roof collapses and wall collapses were common in the areas of strong ground motion (NOAA 1971, SEAONC, 2001). It was clear more restrictive code requirements would be needed.

Beginning with the 1973 UBC, the requirement for positive direct wall ties was introduced and wall anchorage using wood cross-grain bending was expressly prohibited. In addition, continuous crossties were introduced for concrete and masonry walls supported by roof systems. In order to transfer seismic forces from the heavy perimeter walls into the main roof diaphragm, continuous ties or crossties are necessary to drag the load uniformly across the diaphragm depth.

In the 1976, UBC, the concept of subdiaphragms was introduced as an analytical device for transferring forces from the individual wall ties to the continuous crossties (Sheedy and Sheedy, 1992). Instead of creating a continuous tie at every wall anchorage location, continuous crossties can be placed at wider spacing using subdiaphragms. Subdiaphragms are portions of the main diaphragm that span between the continuous crossties and gather the wall anchorage loads and transfer this load to the crossties. Once the load is collected into the continuous crossties it is distributed across the main diaphragm for further distribution to shear walls and frames of the building.

In the 1979 UBC, the wall anchorage design force was increased 50% from $0.2W_p$ to $0.3W_p$ (where W_p is the tributary wall weight) for seismic zone 4. The code improvements of the 1970s were not tested until the 1984 Morgan Hill, California Earthquake. Even though only a few tilt-up buildings saw moderate levels of shaking, it provided the first performance comparison of tilt-up buildings built before and after the 1973 and 1976 code changes. Several 1960s era tilt-up buildings suffered wall anchorage or continuity tie failures. On the other hand, a tilt-up building built to more modern codes saw no structural damage despite the estimated 0.3g-0.4g ground accelerations (EERI 1985).

The Whittier Narrows Earthquake in 1987 tested significantly more tilt-up buildings than the Morgan Hill Earthquake, but forces were still only moderate. Pre-1973 UBC buildings saw failures at the wall in ledger cross-grain bending, cross-grain tension splits at interior diaphragm areas, and diaphragm nailing pulling through the plywood edges, similar to what was observed in the San Fernando Earthquake. In contrast, the tilt-up buildings designed under more modern codes performed much better. Despite the better performance of the more modern codes, there was some concern whether the code provisions were adequate to meet performance objectives under very strong levels of shaking (EERI 1988).

Research on instrumented rigid wall buildings with flexible diaphragms following the 1984 Morgan Hill (Çelebi et al. 1989) and the 1989 Loma Prieta Earthquakes (Bouwkamp, Hamburger, and Gillengerten 1991) indicated that the dynamic response of structures with predominantly solid walls is dominated by the diaphragms, amplifying roof accelerations and corresponding wall anchorage significantly more than previously thought. The recorded amplification of ground accelerations to roof accelerations resulted in the basis of a 1991 UBC revision increasing wall tie forces another 50% to $0.45W_p$ for seismic zone 4 (Sheedy and Sheedy 1992). This higher wall tie force was only required at the center half of the diaphragm span.

The 1994 Northridge, California Earthquake was the first test of modern post-1979 building code tilt-up provisions under very strong shaking. Hundreds of tilt-up buildings were severely damaged with partial roof collapses (CSSC 1995, Brooks 2000). Damage in pre-1973 buildings was not necessarily a surprise, but the numerous wall anchorage failures in post-1973 code buildings were troubling.

The unexpected wall anchorage damage to newer buildings was primarily attributed to two main reasons: inadequate connection overstrength for the roof accelerations and excessive deformation of the wall anchor system (APA 1994, EERI 1996). Light-gauge steel twist straps were especially a problem due to their geometry and limited overstrength (Harris et al. 1998). Research and post-earthquake investigations have shown that rooftop accelerations may be three to four times the ground acceleration, and insufficient overstrength or ductility has been provided in past connection practices (Çelebi et al. 1989, Harris et al. 1998). 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 reliance on ductility. As a result, wall anchorage forces at the roof were increased from $0.45W_p$ to $0.88W_p/1.4=0.63W_p$ (ASD) and the load was applied over the entire span of the diaphragm in the 1997 UBC. Material-specific load factors (1.4 for steel, 0.85 for wood, 1.0 concrete/masonry) were specified to obtain uniform overstrength within the connection to resist the demand from maximum expected roof accelerations of $4C_a$. This approach is well documented in the 1999 SEAOC Blue Book, *Recommended Lateral Force Requirements and Commentary* (SEAOC Seismology Committee 1999, Section C108.2.8.1).

As further evidence of the intent of these wall anchorage provisions, SEAOC stated that the reduced R_p value for nonductile and shallow anchorage in the 1997 UBC does not apply to wall anchorage designed using this overstrength approach (SEAOC Seismology Committee 1999).

No prescriptive deformation limits of the wall tie system have been introduced into the Code, however the compatibility of anchorage system flexibility and diaphragm shear nailing must be considered. Wall anchorage systems with too much flexibility will inadvertently load the wood sheathing edge nailing and either pull the nails through the edge or place the ledgers in cross-grain bending or tension. Pre-manufactured strap-type wall ties are designed to limit the maximum deformation to 1/8 in at their rated allowable load based on legacy ICBO Acceptance Criteria 13 (ICBO 2002), and pre-manufactured hold-down devices using anchor rods could allow even greater deformation. Engineers should contact the device manufacturer for additional deformation information. The hold-down device flexibility is solely within the steel component itself and is additive to other sources of deformation. Additional deformation can be contributed by other anchorage components (e.g. bolts and nails) and installation practices (e.g. oversized holes).

In 2001, the City of Los Angeles adopted a more stringent requirement that limits the deformation of the wall anchorage to 3/8" under $3F_P$ loading, with no more than 1/8 in occurring in the steel connector itself. The 3/8 in deformation limit includes contributions from slip in nails, bolts, or screws; wood shrinkage; deformation of steel, concrete, and wood components; and inelastic deformation in the anchor connector between the wall and the attached framing member (LADBS 2002). The intent is to rationally limit the deformation under maximum expected wall tie forces at the roof level ($3F_P$) to a dimension equal to the minimum nail distance to the sheathing edge (3/8 in). It is recommended that new editions of the building code limit seismic wall anchorage deformation using a similar approach.

Current Wall Anchorage Provisions

ASCE 7-05 Section 12.11.2.1 governs wall anchorage design for most of the tilt-up buildings in California, where Seismic Design Category C and higher applies for bearing walls. In the development of ASCE 7-05, the intent was to maintain the same wall anchorage forces as in the 1997 UBC for flexible diaphragms in high seismic zones. Substituting $C_a = 0.4S_{DS}$ (BSSC 2004b), it can be confirmed that ASCE 7-05 Eq. 12.11-1 is generally equivalent to the 1997 UBC.

The wall tie forces of $F_P=0.8S_{DS}IW_P$ for flexible diaphragms are double the normal wall design force in Section 12.11.1 and four times the typical tilt-up building base shear to account for the expected rooftop amplification associated with flexible diaphragms. A steel material load factor of 1.4 is applied to the wall tie forces to reach necessary levels of material overstrength as in the past 1997 UBC Section 1633.2.8.1. This steel component factor applies to the wall anchorage system, and because the wall anchorage system is defined to include the wall ties, subdiaphragm, and continuity ties, the 1.4 factor applies to all steel components and members in these subsystems. Steel components governed by wood capacity, such as nails and bolts in shear, are not subject to the 1.4 multiplier because of the greater material overstrength available from wood (Harris et al. 1998). The 0.85 material load factor for wood components in the 1997 UBC Section 1633.2.8.1 is neither a part of the ASCE 7-05 nor the 2006 IBC provisions. Thus ASCE 7-05 will result in a more conservative design of the wood portion of the wall anchor system than the 1997 UBC.

To summarize, the ASCE 7-05 implements 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 of various materials. Through an unrelated parallel effort, ACI 318-05 has inadvertently combined the elevated force levels of the overstrength approach in ASCE 7-05 with new ductility requirements for seismic anchorage in ACI 318-05 Appendix D Sections D.3.3.4 and D.3.3.5. These ductility requirements were inadvertently added on top of the elevated anchorage force levels in conflict with the original intent of the ASCE provisions.

Furthermore, 2006 IBC Section 1908.1.16 allows an additional 2.5 load factor on top of already elevated anchorage forces in ASCE 7-05 in lieu of the ACI ductility requirement. The IBC 2.5 load factor or ACI ductility requirement used on top of elevated wall anchorage provisions of ASCE 7-05 Section 12.11.2.1 is not appropriate and may be difficult to achieve in tilt-up wall anchorage design.

At non-flexible diaphragms, wall anchorage provisions are provided under ASCE 7-05 Section 12.11.1 resulting in F_P=0.40$S_{DS}IW_p$. This value for F_P is significantly lower than that in flexible diaphragms due to the lesser amplification associated with that construction. Seldom are tilt-up wall panels defined as nonstructural walls (walls that have less than 200 plf superimposed gravity load and are also not lateral force-resisting walls). Anchorage of nonstructural walls to non-flexible diaphragms is designed under ASCE 7-05 Section 13.3 with additional provisions of Section 13.5.3. At nonstructural walls supported by flexible diaphragms, a footnote at the bottom of Table 13.5-1 references Section 12.11.2 in lieu of the provisions of Section 13.3, making anchorage of structural and nonstructural walls to flexible diaphragms the same.

Wall anchorage forces act in compression as well as tension. Panelized wood roof systems by their very nature are not erected tight against the perimeter wall ledger, leaving a small gap to potentially close during seismic compression forces. This gap is properly a result of casting and erection tolerances of construction. Strap-type wall anchors that have yielded and stretched under tensile forces are vulnerable to buckling and low-cycle fatigue as the gaps close. Cast-in-place anchor rods used in connectors can be checked for compression, but it is important to provide an additional nut against the interior wall surface to prevent the anchor punching through the wall. At steel ledger conditions, often wall anchorage is achieved with steel angle straps that are bolted to the roof structure and capable of resisting compressive forces. Although there have been no failures of wall panels collapsing into buildings, consideration of compressive forces will better maintain the integrity of the wall anchorage tie for tension forces.

Connections that are loaded eccentrically or are not perpendicular to the wall are required to be investigated for any bending and all resulting force components induced by the configuration. The bending induced by single-sided connections combined with the wall tie axial load may overstress the attached wood roof member and be a source of potential failure (Hamburger and Nelson 1999). It is recommended that wall tie connectors be applied symmetrically where possible.

Failures of beam seats or spalling of pilasters have occurred in recent past earthquakes (EERI 1996). Where pilasters occur, ASCE 7-05 Section 12.11.2.2.7 requires consideration of the force concentration due to the pilaster stiffening effect on the wall out-of-plane. The anchorage force at the top of the pilaster is determined by considering two-way bending action of the wall panel. This concentrated force is applied directly to any framing member anchored to the top of the pilaster. Reduction of the typical wall anchorage force elsewhere along the wall is not permitted.

Anchorage to Wood Diaphragms. As in previous codes since the 1970s, wall anchorage to wood roof systems is not allowed to depend upon cross-grain bending, nailing in withdrawal, or diaphragm sheathing in tension. Wall anchorage loads are transferred into the main diaphragm with subdiaphragms and continuous crossties. These provisions are a direct result of the poor performance during the 1971 San Fernando Earthquake.

Subdiaphragms are provided under ASCE 7-05 Section 12.14.7.5.1 as an analytical device to provide a rational load path for wall anchorage. Subdiaphragm aspect ratios are limited to 2½ to 1, and this provides sufficient stiffness that the independent deflection between the subdiaphragm and the main diaphragm may be ignored. Tilt-up warehouse

buildings today often have large column spacing of 50, 60, even 70 feet, resulting in very large subdiaphragm spans and corresponding subdiaphragm depths. Pursuant to investigations of damages to wood diaphragms after the 1994 Northridge Earthquake, the joint task force recommended continuous ties at specified spacing to control cross grain tension in the interior of diaphragm and subdiaphragm shear limited to control combined orthogonal stressed within the subdiaphragm. As a result, Los Angeles City and Los Angeles County jurisdictions have taken a conservative approach by limiting spacing of continuous ties to 25 ft and subdiaphragm shears to 300 plf. In a recent update of the Los Angeles Regional Uniform Code Program (LARUCP) as part of adoption process of the 2007 California Building Code, the continuous tie spacing limit is increased to 40 ft and shear value increased to 75% of the code-allowable diaphragm shear in the determination of subdiaphragm depth. The revisions were based on improved performance and standards for diaphragm construction today. (2007 LARUCP.)

The benefit of the 300 plf upper bound on subdiaphragm shear strength is the reserve capacity available for orthogonal effects from seismic loading. Because subdiaphragms are a part of the main diaphragm, they are theoretically subject to shear loads from both orthogonal directions. Consideration of orthogonal loading effects in diaphragm shears is not normal practice today, however this approach may be more rational in lieu of arbitrary shear capacity limits. One such approach worth considering is to limit subdiaphragm shears to 1.0/(1.0+0.3) = 77% of their allowable diaphragm shear value, reserving the remaining 23% for orthogonal effects as discussed in the 2003 NEHRP Commentary Section 4.2.2 (NEHRP 2003).

Research indicates that the dynamic amplification associated with flexible diaphragms amplifies the wall anchorage forces, but this increase is limited by yielding of the roof diaphragm. Under low levels of ground motion, roof diaphragms remain elastic and amplify ground forces 3 to 3½ times, but under strong ground motion levels the amplification is reduced to approximately 2½ times due to nonlinear behavior. This reduction in amplification is beneficial to the wall anchorage system, because system failure is now initiating in the more ductile diaphragm instead of the wall anchorage components. However, this assumes the diaphragm is not excessively conservative. Because tilt-up buildings are often long and narrow, diaphragm designs are more governed by forces in the transverse direction, resulting in conservative overstrength in the longitudinal direction. This results in more elastic diaphragm behavior in the longitudinal direction, and thus larger wall anchorage force amplifications at the narrow ends of the building, with forces possibly exceeding the level of 1.2g anticipated in the current code provisions (Harris et al. 1998).

Anchorage of Walls to Metal Deck Diaphragms. Although less common in California than panelized wood sheathing, flexible metal deck diaphragms (without fill) are becoming more common in tilt-up construction in seismically active areas. When designed properly, metal decking can assist in providing wall anchorage and eliminate the need for subdiaphragms by acting itself as the continuous crossties. Important detailing issues must be carefully considered.

Metal deck can only provide continuous crossties parallel to the deck span direction. ASCE 7-05 Section 12.14.7.5.3 specifically prohibits use of metal deck perpendicular to the direction of span for continuity, because the deck flutes will stretch out and flatten. Where the decking is spliced, a common structural member is necessary to receive the attachment from both deck panels. In common steel joist (truss) systems with double top chords, it is necessary that both deck panels be attached to the same individual top chord half, otherwise crosstie loads will be inadvertently transferred through the steel joist (truss) top chord separation plate or web welding, depending on joist web configuration. Another concern at the deck panel splice and direct ledger attachment is the weld tear-out through the metal deck. Proper deck gauge and puddle weld edge distance must be maintained for adequate wall anchorage strength. A better approach is to provide steel angles perpendicular to the wall to transfer wall anchorage into the diaphragm, similar to a wood roof system approach.

Another challenge with metal deck diaphragms is the need for thermal expansion joints. Metal deck roof diaphragms are much more vulnerable to temperature swings than wood diaphragm systems, and with the trend towards larger roof dimensions, thermal expansion joints become more likely. However, these expansion joints interrupt the continuity of the wall anchorage system (crossties) and thus create several independent structural units to be analyzed separately. The wall anchorage forces must be fully developed into the main diaphragm and transferred to

the applicable shear walls before reaching the expansion joint. This results in larger diaphragm shears when compared with wood diaphragms without expansion joints.

If the metal deck is expected to carry wall anchorage forces, it must be investigated for tension and compression axial loads in conjunction with acting gravity loads. The axial compression loads are associated with inward wall forces and require a special axial/bending analysis of the decking. The *North American Specification for Design of Cold-Formed Steel Structural Members* (AISI 2001) provides design criteria for the decking, and the Structural Steel Education Council (Mayo 2001) illustrates one approach for this wall anchorage.

Anchorage to Roof Framing. Whether using a panelized wood sheathed roof or a metal deck roof, steel trusses or joists are now the most common roof framing members in tilt-up buildings in California. This trend began in the early 1990s when rising timber prices increased the cost of traditional wood roof systems. In terms of wall anchorage, the use of steel truss framing is advantageous because steel joists are attached by direct welding at the steel wall ledger and at interior crosstie splice locations. This eliminates the connection deformation or "stretch" problems that contributed to cross-grain bending and plywood edge nailing pulling out of sheathing edges during past earthquakes. Steel joist systems are typically designed by specialty engineers in association with the manufacturer, and the building design engineer is responsible for providing axial wall tie and continuity tie loads to the manufacturer along with information stating which load factors if any have already been applied (IBC Section 2206.2). In conditions where axial loads are transferred through the joist seat, it must be made clear to the manufacturer so that the seat strength can be checked also. There are limits to the amount of load that manufacturers can transfer through these joist seats. The explosive growth of the steel joist system in tilt-up buildings has occurred since the 1994 Northridge Earthquake, and it remains to be seen what level of performance may occur in these buildings when subject to severe shaking.

An additional result of the 1994 Northridge Earthquake was the passage by the City of Los Angeles of a new tilt-up retrofit ordinance. After estimating that one third of the nearly 1200 tilt-up buildings in the San Fernando Valley suffered significant damage in that earthquake, Los Angeles unveiled an ordinance that was developed in conjunction with SEAOSC to require wall anchorage and continuity ties in existing pre-1976 tilt-up buildings. A similar ordinance was adopted in Los Angeles County and in other jurisdictions not long after. This ordinance does not attempt to force older tilt-up buildings to comply with current code requirements, but instead aims to obtain levels of performance consistent with acceptable minimum life safety. Additional earthquake hazard reduction information is also available from other publications (LA City 2002; LA County 2002, ICBO 2001, SEAONC 2001).

Diaphragms

The most common roof system used in tilt-up construction today in California is the hybrid roof. This consists of wood structural-use panels such as plywood or oriented strand board (OSB) nailed to wood nailers factory installed to the top chord of open-web steel joists. Current tilt-up development trends include larger and taller buildings with more clear-space and clear-height to facilitate warehousing and distribution. These trends are placing more demand on the wood roof diaphragms to span farther horizontally with higher shear stresses. From the 1967 UBC and through the current 2006 CBC, the maximum allowable shear in horizontal wood diaphragms is 820 plf per IBC Table 2306.3.1. The 820 plf table maximum may be exceeded with a special high-load wood diaphragm with multiple rows of fasteners per IBC Table 2306.3.2. In large tilt-up buildings, engineers have relied upon these high-load diaphragm values with shear capacities up to 1800 plf. First introduced as an ICBO Evaluation Service ER document, these high-capacity wood diaphragms are now incorporated into the IBC. Special inspection is required, however many special inspectors do not have experience or certification with this type of inspection. If some doubt exists as to the qualifications of the Special Inspector, it is recommended that a preconstruction meeting be held to clarify the inspection issues.

Less common in California tilt-up buildings is the use of metal deck as the roof diaphragm. These diaphragms are capable of higher shears than wood diaphragms, but concerns of thermal expansion often limit the diaphragm width

by introducing expansion joints, as noted above. Metal deck diaphragms are capable of reaching over 3000 plf allowable design values with heavy gauge material and special attachments.

The performance of tilt-up building diaphragms will be more critical now as the problems associated with wall anchorage are overcome. With wall anchorage and collector design forces factored up to maximum expected levels, and with perimeter shear walls often consisting of solid wall panels providing excessive lateral strength, ductile yielding of the roof diaphragm is thought to be the likely failure mode in new structures. Unfortunately, damage to these diaphragms is not easily observed until significant separations or partial collapses occur.

Diaphragm Chords. Diaphragm behavior is similar to a flat beam, with diaphragm chords acting as beam flanges. With larger roof diaphragms, higher chord forces also occur. In the past, structural chords typically consisted of special reinforcing bars embedded in the wall panels near the roofline connected together with welding across the panel joints. With the disappearance of cast-in-place pilasters or stitch columns in the 1970s, the new dry panel joints resulted in chord connections that were more vulnerable to settlement and concrete shrinkage strains. Distress at chord connections across panel joints was taking on the form of concrete spalling and weld-breaks even without earthquakes.

Prior to the late 1980s, reinforcing chord bars were generally of ASTM A615 Grade 60 material, which was vulnerable to weld embrittlement failures (in the welded splices) due to improper preheating. Broken chord connections at these welds were observed in the 1987 Whittier Narrows Earthquake (EERI 1988). Beginning with the 1985 UBC, ASTM A706 reinforcing was introduced for welded locations and seismic frame reinforcing because of the tight controls on carbon equivalency and yield limits for that material. ASTM A706 requires less preheating and results in fewer weld embrittlement problems, and it is thus recommended for welded chord reinforcing. IBC Section 1908.1.5 discourages welding of ASTM A615 reinforcing by requiring a report of material properties from the producer to establish proper preheating. ASTM A706 does not have this requirement.

Steel ledger systems began to appear in the mid 1980s, and these steel channels or angles were welded together across the panel joints to function as diaphragm chords. This circumvented the weld embrittlement problem with reinforcing bar connections, and steel ledgers were observed to have performed very well in the Whittier Narrows earthquake (EERI 1988). Steel ledgers are commonly used today for supporting the roof system and acting as diaphragm chord reinforcing.

Another source of earthquake damage in chord connections is the condition of skewed or angled wall connections. With tilt-up buildings now frequently being placed on irregularly shaped lots, building plans are no longer simply L-shaped or rectangular. Often corners are clipped or angled to accommodate adjacent property lines or easements. Tensile chord forces in flexible diaphragms must resolve themselves around the skewed wall connection, and these forces result in an inward or outward thrust component. This component is seldom resolved properly with a connection into the main diaphragm, and it results in chord reinforcing or steel ledgers being damaged (EERI 1988, EQE 1989).

In large diaphragm systems, the numerous continuity ties required for the wall anchorage system will behave inadvertently as a collection of chord elements across the diaphragm. Based on strain compatibility, the continuity ties all assist in providing a collective chord that distributes chord forces over a larger portion of the roof structure (Lawson 2007c). This approach is closer to the actual behavior of the diaphragm and can result in substantially lower forces in each chord member. For simplicity, engineers typically model chords as a single element at the perimeter of the building.

Diaphragm collectors. Significant collector forces are often found in tilt-up construction due to the large open interior spaces, with collectors dragging forces to re-entrant corner shear elements or interior braced frame elements. Transferring the loads from roof framing collectors into the concrete wall panels can be challenging, especially at re-entrant corners that may have two intersecting collectors. Typically, the collector is connected to an embedded steel plate with drag reinforcing used to distribute the load into the shear wall line. The collector loads are normally

dragged across panel joints in order to distribute the collected diaphragm load into sufficient wall length. The collector load in the roof structure is subject to the special load combinations referenced in ASCE 7-05 Section 12.4.3.2.

In-plane Diaphragm Deflection. In tilt-up shear wall buildings, diaphragm deflection is computed for the design of building separations and deformation compatibility. The in-plane shear wall drift is typically insignificant compared with the diaphragm deflection and is usually ignored. Also ignored is wall panel out-of-plane deflection when considering building separations. The estimated deflection of plywood diaphragms may be obtained from analysis per IBC Section 2305.2.2. Research indicates that under seismic loading, plywood diaphragms are stiffer than indicated by the conventional diaphragm displacement formulas and subject to less displacement amplification compared to force amplification (Harris et al. 1998). In larger roof systems, many of the continuity ties act inadvertently as collective chords reducing the actual diaphragm deflection (Lawson 2007c). Use of a single chord design model will typically be conservative; however, dynamic diaphragm modeling being used in research could overestimate the diaphragm period and obtain unconservative results. Metal deck diaphragm deflections are computed using formulas furnished in a particular product ICC Evaluation Report.

In tilt-up buildings, diaphragm deflection results in the columns and perpendicular walls rotating about their bases due to diaphragm translation at the top. Assuming the columns and walls were modeled with pinned bases during their individual design, this base rotation is permitted to occur even if some unintentional fixity exists. Unintentional fixity may be the result of standard column base plate anchorage or wall-to-slab anchorage combined with any wall-to-footing anchorage. ACI 318-05 Section 14.8 requires slender tilt-up walls to be tension-controlled and limits the factored vertical concrete stress to $0.06f_c$'. This allows panels to better accommodate any localize yielding at the base while continuing to carry the vertical loads.

Diaphragm deflection is not normally included in the story drift limits of ASCE 7-05 Section 12.12.1. Story drift limits were developed with the intent to limit the deformation of the basic vertically-oriented elements of the seismic force-resisting system. In tilt-up buildings, these vertical elements deflect very little in-plane, with all of the translation occurring at other elements. Story drift limits do not apply to diaphragm deflection.

Over the past decade, most practitioners have calculated diaphragm deflections only for the purposes of building setbacks from property lines or other adjacent buildings, such as that required in ASCE 7-05 Section 12.12.3. With the larger and more flexible diaphragms being built today, IBC Section 2305.2.2 is becoming more important to consider:

> Permissible deflection shall be that deflection up to which the diaphragm and any attached load distributing or resisting element will maintain its structural integrity under design load conditions, such that the resisting element will continue to support design loads without danger to occupants of the structure.

By intention, this language is not clearly defined, with the approach left much to the engineer’s own judgment. In low-rise concrete or masonry buildings, excessive deflections in horizontal diaphragms can cause overall instability in walls and columns from the *P*-delta effect. Gravity load-bearing walls and columns, when subjected to horizontal translation at the top, will begin to induce a horizontal thrust into the diaphragm, further exacerbating the deflection. Although it was not originally intended to be used to evaluate diaphragm deformations, ASCE 7-05 Section 12.8.7 can be used as to investigate roof stability under *P*-delta effects.

Diaphragm Re-entrant Corners. Unfortunately, tilt-up standard practice has often been to ignore the stiffness at short re-entrant corners, with the false belief that the short re-entrant walls were not designated shear walls and would somehow accommodate the diaphragm movement. This mistaken assumption reflects the over-simplification of design made by engineers when analyzing these very flexible diaphragms. Even though deformation compatibility provisions have been in the code since the 1976 UBC, designers have often ignored this provision in tilt-up buildings. Deformation compatibility problems have been documented in the 1984 Morgan Hill Earthquake

(EERI 1985), the 1987 Whittier Narrows Earthquake (EERI 1988), and the 1989 Loma Prieta Earthquake (EERI 1990), where diaphragm drift tore apart roofs at unintended stiff wall elements, such as at re-entrant corners and where fin walls were located, and racked interior partition walls in contact with either the roof or tilt-up walls.

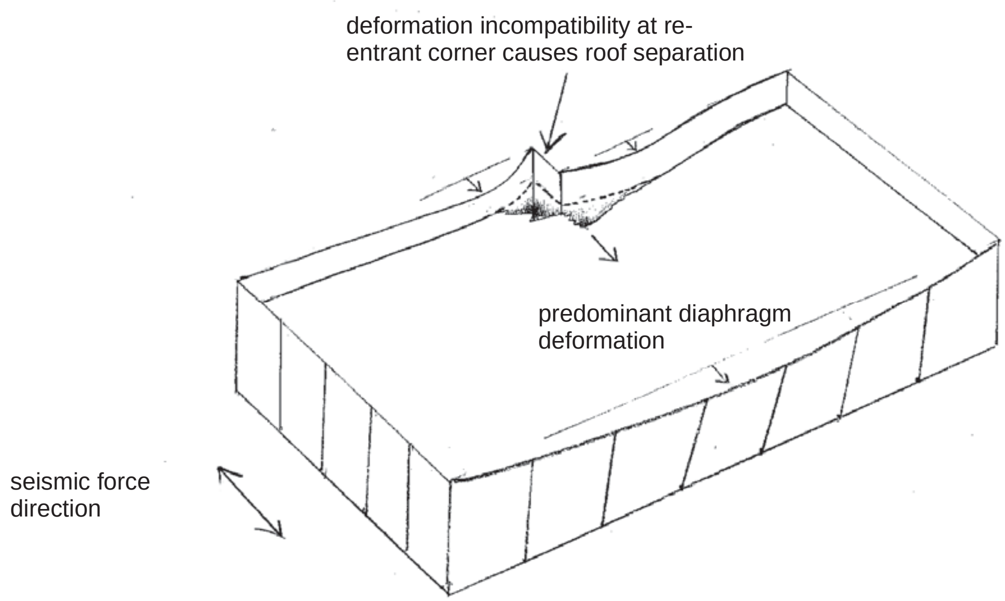

Figure 5. Effect of Shear Wall at Re-entrant Corner on Diaphragm Deformation

Today, tilt-up buildings are receiving more attention in terms of architectural design, with city planners and architects articulating the formerly large flat wall surfaces in an effort to make industrial parks and warehouses more aesthetically pleasing. Often this results in numerous re-entrant corners, buttresses, fin walls, or other stiff elements inadvertently resisting the diaphragm drift. The most direct approach to address re-entrant corners is to design shear walls along each line of the corner, thus eliminating any deformation incompatibility issue. This solution unfortunately requires long drag strut or collector lines, and the configuration of the re-entrant corner may not be suitable for introduction of a shear wall at that location. Another approach could be to install an interior shear wall or braced frame element near the re-entrant wall to minimize the diaphragm deflection at the re-entrant wall.

In situations where the re-entrant corners or other stiff elements occur at locations of large diaphragm drift, several other options may be available. Where the re-entrant corner or stiff element depth is very small, an engineer can investigate the option of allowing the element to rock or begin to overturn in a controlled manner. The engineer can calculate the amount of force necessary to rock the stiff element and conservatively attach a diaphragm strut to develop that rocking force into the main diaphragm. This solution prevents roof separations due to stiffness incompatibility, but the designer must pay careful attention to detailing to ensure rocking panels are not inadvertently joined together or overly anchored to the foundation unless the resulting larger overturning force is accounted for. Supplemental vertical supports for major members framing into such re-entrant corners should be considered.

When investigating the rocking of a panel, it is possible that the panel itself is perforated to the point that the wall piers or frame-like members will fail in bending before the rocking relief takes place. In this situation, the wall piers

must maintain their integrity and are detailed under ACI 318-05 Section 21.11 for frame members not proportioned to resist forces induced by earthquake motions. The re-entrant panel is still connected to the roof structure to develop the load into the diaphragm necessary to achieve the pushover capacity of the piers.

Other solutions may involve the use of seismic isolation or expansion joints in the roof structure. Depending upon the configuration, the offending re-entrant elements can be isolated from the main diaphragm movement, however this requires the isolated portions of the building to remain stable on their own.

In ASCE 7-05, there are no provisions that indicate the amount of plan offset or re-entrant depth that these re-entrant corners can reach before requiring investigation. In 2000 IBC Section 1617.4.4.2, shear wall line offsets up to 5% of the building dimension are considered small enough in flexible diaphragms to allow the designer to neglect the offset and design it as one single shear wall line when distributing horizontal diaphragm shears. In the 2003 IBC, this language was removed when ASCE 7-02 became the reference document for this section. ASCE 7-05 does not have this language, other than identifying plan structural irregularities as 15% of the building dimension regardless of diaphragm flexibility (ASCE 7-05 Table 12.3-1). Because of the more critical nature of re-entrant corners in flexible diaphragms, it is unconservative to ignore re-entrant corners measuring up to 15% of the building dimension. It is recommended that engineers address the deformation compatibility issue associated with re-entrant corners where the out-of-plane wall offsets are more than 5% of the diaphragm span dimension perpendicular to the direction of lateral load, as shown in Figure 6.

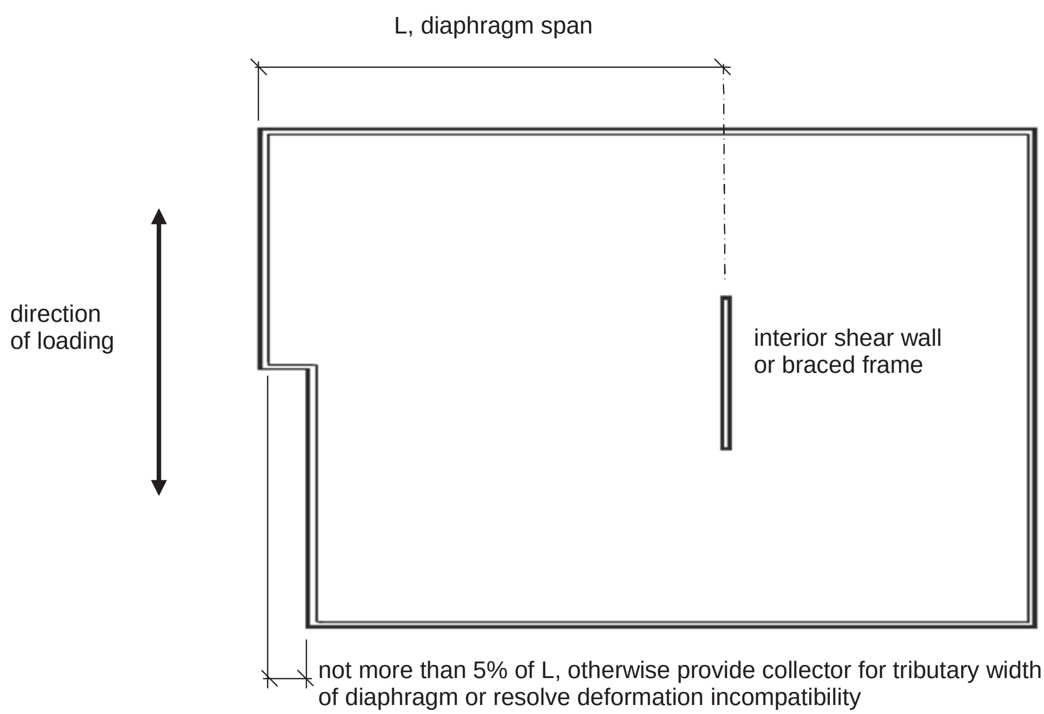

Figure 6. Plan of Typical Tilt-up Building
Illustrating Guidelines for Re-entrant Corner Considerations in Flexible Diaphragms

Multiple-Story Diaphragm Compatibility. Originally, tilt-up buildings were one-story warehouses with an occasional woodframe mezzanine. As these buildings gained greater acceptance for office environments, the mezzanines grew to full second floors and in some instances third floors. Higher-end office buildings are now using

concrete floor systems over metal deck, while the roof system remains a flexible panelized wood roof or metal deck roof.

With concrete panels extending full height continuously past the upper floor, diaphragm deflection incompatibilities between the roof and floor diaphragms can lead to panel damage or anchorage failure. This type of damage was first observed in the 1989 Loma Prieta Earthquake where a full-height wall panel was anchored to both the roof and second floor. The panel experienced cracking along the second floor level, indicating the initiation of a horizontal hinge (SEAOC 1991).

For analytical purposes, engineers normally ignore wall continuity and assume the panel hinges at the intermediate floor lines, thus anchoring the out-of-plane wall load based on simple tributary wall areas to the floor and roof levels. In this situation, some cracking and hinging of the panel at the intermediate floor line is anticipated under strong shaking levels, and this is acceptable as long as the axial gravity loads are still characteristically small (limited by ACI 318-05 Section 14.8.2.6) and the wall panel is sufficiently flexible out-of-plane. A worse scenario is if the wall is excessively stiff out-of-plane and the relative roof and floor movements pry the wall anchorage loose causing a localized collapse. In conditions where diaphragm drift significantly varies from floor-to-floor or floor-to-roof, the designer should investigate the wall anchorage capacity for this additional effect by analyzing deformation compatibilities or designing the stability of the floor to be independent of the concrete wall system.

Is Tilt-up Construction Precast or Cast-in-Place?

In the past, tilt-up engineering has generally followed design and detailing provisions as for monolithic concrete. Precast concrete buildings were traditionally considered as structures comprised of numerous small concrete members cast at an off-site plant and transported to the jobsite for assemblage. Individual beam, column, plank, and narrow wall members are traditional precast elements, and ACI 318 precast concrete provisions were developed with that construction type primarily in mind. Generally, the joining of beams to columns for frame resistance or the coupling of narrow wall elements together for composite shear wall resistance represents traditional precast concrete resistance to seismic forces. In traditional precast construction, the individual element has little or no lateral resistance on its own, but relies upon the assemblage to achieve lateral resistance.

In contrast to traditional precast, concrete tilt-up construction consists of panels that are individually stable wall elements that seldom require coupling devices for composite action. Penetrations within walls are surrounded by deep beams above which in turn are monolithically cast to wall pier elements at the sides. The only discontinuities are at the vertical panel joints, often twenty or thirty feet apart, and at the panel-to-footing interface. Historically, the in-plane performance and ductility of this lateral force resisting system is closer to cast-in-place construction than traditional precast.

The ACI 318-05 defines precast concrete in Section 2.2 as a "Structural concrete element cast elsewhere than its final position in the structure." Under this broad definition, tilt-up concrete construction could be considered as site-cast precast. In fact, the tilt-up concrete construction method was first discussed in ACI 318-95 in the Commentary for precast concrete stating, "Tilt-up concrete construction is a form of precast concrete."

In ASCE 7-05 Table 12.2-1, "Design Coefficients and Factors for Seismic Force-Resisting Systems," there are now three categories under the heading for Bearing Wall Systems and for Building Frame Systems that potentially apply to tilt-up buildings as a form of precast concrete construction. These are Ordinary Precast Shear Walls, Intermediate Precast Shear Walls, and Special Reinforced Concrete Shear Walls. The category of Intermediate Precast Shear Walls is new and represents a transition in detailing and expected performance between ordinary and special systems. ACI 318-05 Commentary 21.1 indicates that the Intermediate Precast Shear Wall system is equivalent to a Cast-in-Place Ordinary Reinforced Concrete Shear Wall. However, ASCE 7-05 places height limits on the Intermediate Precast system in Seismic Design Categories D, E, and F, whereas Ordinary Reinforced Concrete Shear Walls are not permitted in Seismic Design Categories D, E, and F per ASCE 7-05; Intermediate Precast Shear Walls are permitted recognizing their additional wall pier detailing requirements and height limits. 2006 IBC

1908.1.8 gives detailing requirements for a special wall pier when design is based on ACI 318 Sec. 21.7, while IBC 1908.1.13 gives requirements for wall pier detailing when design is based on ACI 318 Sec. 21.13. As discussed later under Wall Pier and Shear Wall Classifications, the provision prescribed under IBC 1908.1.8 was introduced by SEAOC in legacy code for high seismic regions (SDC D, E or F), while the later provision prescribed under IBC 1908.1.13 has been introduced by others for lower seismic region (SDC C.)

With the lack of the word "precast," the applicability of the Special Reinforced Concrete Shear Wall category to tilt-up buildings may not be immediately obvious, but a quick reference to ACI 318-05 will clarify this application. The intent of ACI 318-05 toward the proper classification of Structural Walls can be obtained by reviewing the provisions in conjunction with the Commentary. Chapter 21 of ACI 318-05, Special Provisions for Seismic Design, Section 21.1 - Definitions, provides the following guidance. Structural Walls are defined as "Walls proportioned to resist combinations of shears, moments, and axial forces induced by earthquake motions. A shear wall is a structural wall. Structural walls shall be categorized as follows." A Special Precast Structural Wall is defined as "A precast wall complying with the requirements of 21.8. In addition, the requirements for ordinary reinforced concrete structural walls and the requirements of 21.2.2.3, 21.2.3 through 21.2.7, and 21.7 shall be satisfied." The commentary for this definition states, "The provisions of 21.8 are intended to result in a special precast structural wall having minimum strength and toughness equivalent to that for a special reinforced concrete structural wall of cast-in-place concrete." Therefore this establishes that a Special Precast Structural Wall is equivalent to a Special Reinforced Concrete Structural Wall.

Development of precast concrete seismic design provisions has been based on extensive research which resulted in development of acceptance criteria described in 2003 in FEMA 450-2, Commentary section 9.6 (BSSC 2004a). Hawkins and Ghosh (2004) provide information on testing research on this topic. Much of the research work has been directed toward investigating the seismic performance of traditional precast concrete structures with improved connections. The landmark research associated with the PRESSS (Precast Seismic Structural Systems) Research Program has greatly influenced the specific ACI 318 seismic provisions for precast concrete systems. ACI 318-05 Sec. 21.8 provisions on special structural walls constructed using precast concrete, however, was not written for tilt-up wall construction (Ghosh and Hawkins 2006).

The development of the special precast concrete system was separate from the over fifty-year development of the concrete tilt-up system. The current code language is ambiguous, because ACI 318 Chapter 21 encourages ductile detailing, including confinement reinforcement and development of tensile reinforcement in high-seismic regions. SEAOC believes a new system approach for tilt-up system is needed. During the interim, SEAOC affirms proper load path and ductile detailing practice be followed in the design and detailing of tilt-up panels.

Shear Distribution in Walls Loaded In-Plane

Complex panel configurations and the wall panels with extensive perforations are relatively recent developments. Originally, designers had ample amounts of solid wall panels to use or "designate" as acting as shear walls. Often, there was enough overstrength along a wall line that little attention was paid to exact force distribution among the wall panels. It was common to simply divide the total wall line shear equally into each panel or proportional to panel length, accounting somewhat for the openings. Even though past codes required that forces be distributed in proportion to element stiffnesses, engineers justified designs by demonstrating adequacy of the collective wall line as a whole. A series of individual panels modeled together with multiple opening configurations and numerous panel joints made modeling too complex without computers for the average engineer.

Today, with computer usage common and essential in the office, engineers have enough computing power to better distribute shear loads along complicated shear wall lines in proportion to individual panel rigidities as described by ASCE 7-05 Section 12.8.4, considering both shear and flexural stiffnesses. Accurate distribution of in-plane shear forces along designated shear wall lines has become quite complex as buildings use more complex panel configurations with numerous openings. The classic conventional rigidity analysis often used for walls in poured-in-place concrete or masonry construction is not necessarily accurate for tilt-up walls unless each panel joint is also considered. The combination of repeating panel joints, various opening arrangements, thickened wall sections,

sloping roof heights and partially cracked concrete properties result in a very difficult analysis if distribution accuracy is paramount. Normally, the distribution may be simplified by assuming average roof heights, average panel thicknesses, and uncracked concrete properties.

Recognizing that stiff shear wall elements could become overloaded, crack, and redistribute their loads to other more flexible wall pier elements (see wall pier discussion below), wall piers are required to be detailed to ensure flexural yield failure unless the sum of the shear wall stiffnesses is at least six times greater than the sum of wall pier stiffnesses (IBC 1908.1.8).

Another possible approach used to further simplify the distribution of in-plane shears is to designate solid panels along a wall line as the primary shear walls for the total wall line shear. This simplifies the distribution by ignoring the more flexible panels and transferring the entire seismic load to a few stiffer and stronger panels through a collector. By ignoring portions of the concrete wall line, these ignored elements may be subject to the provisions of ACI 318-05 Section 21.11 for concrete members not designated as part of the lateral force-resisting system. The special detailing of these "ignored" members will provide more ductile behavior in the event that the solid walls crack or rock under overload and redistribute the seismic forces.

A method used by some engineers can be described as the "flexible-link" approach. This method assumes that the chord connections between panels can be designed to contain enough stretch or flexibility to effectively isolate or buffer the individual panels from each other (Eddington 1990, Brooks 2000). Truly isolated panels would theoretically only see seismic forces from the tributary diaphragm length in contact with the panel and forces from the panel seismic self-weight. The flexible link is a panel-to-panel chord connection that has significantly more flexibility than the differences in flexibilities between panels, so that the chord stretch deformation dominates any relative drift differences between panels. It is common in tilt-up construction to prevent the bonding of the chord reinforcing (or slot the continuous steel ledger bolting) in the vicinity of panel joints to help relieve thermal and shrinkage strains that develop. With this approach, the unbonded chord length that crosses a standard ½ in or ¾ in panel joint is considered the flexible-link buffer. Of course this flexible link must still have sufficient stiffness to be effective against chord forces without excessive diaphragm deflections, and analysis complications occur when collectors at re-entrant corners apply large seismic drag loads to the end panel of a wall line.

As just discussed, there are several available methods to distributing seismic shears within the shear wall system. ASCE 7-05 Section 12.8.4 states that seismic story shears shall be distributed to the various vertical elements of the seismic force-resisting system based on the relative lateral stiffness of the resisting elements and diaphragm. The 2003 NEHRP Commentary (Section 5.2.4) states, "Reasonable and consistent assumptions regarding the stiffness of concrete and masonry elements may be used for analysis in distributing the shear force to such elements connected by a horizontal diaphragm." Regardless of which assumptions are used to distribute in-plane shears, adequate life-safety protection is expected under the actual distribution as long as a rational distribution approach is used and the detailing requirements associated with wall piers and frame elements are implemented. This is an area that could benefit from further research.

Wall Pier and Shear Wall Classifications

Traditionally, tilt-up buildings contained many solid wall panels that easily fit the definition of shear walls. However, as greater architectural demands on tilt-up buildings pushed doors and window closer together and closer to panel joints, the remaining wall piers became narrower and more frame-like. Building codes gave little guidance in classifying whether narrow wall segments were better judged as frames or shear walls, and building departments began to see tilt-up wall-frame like structures being designed under the more relaxed shear wall requirements.

In defining the different types of lateral force-resisting systems, concrete wall-framed type systems such as tilt-up buildings with deep spandrels above repeating sizeable openings have not been recognized and have no assigned seismic response *R*-factor. Codes prior to 1991 did not include lower-bound limits on shear wall lengths, and did not adequately cover the design and detailing of slender and narrow shear wall segments. Observed earthquake damage in cast-in-place and precast shear walls repeatedly showed distress due to short-column effects in narrow wall piers

and showed a need for adequate transverse reinforcement. SEAOC Seismology Committee introduced a code change that was adopted into the 1991 UBC. Most of the original provisions sustained transition from UBC to 2006 IBC in defining regions that need special transverse reinforcement in wall piers in order to confine longitudinal wall reinforcement when subjected to in-plane seismic shear force. While IBC Section 1908.1.8 includes the wall pier provisions from the 1997 UBC, similar language has not been incorporated into ACI 318-05.

Wall panels supporting deep spandrels are similar to columns supporting discontinuous shear walls. When a wall segment with the length-to-thickness aspect ratio exceeds 2.5 and the compressive axial force including earthquake is less than $A_g f'_c/20$, the wall pier design provisions should be met. The design shear force, V_e , should be determined by considering the probable flexural strength, M_{pr} , of the wall pier, based on the reinforcement tensile stress of $1.25f_y$ and a capacity reduction factor equal to one, and dividing by the clear height of the opening.

The minimum length-to-thickness aspect ratio was chosen to ensure flexural behavior of the wall segment. A wall segment that does not meet the prescribed aspect ratio may be designed as flexural frame members under ACI 318-05 Section 21.3 if lightly loaded axially and if it meets certain geometric limitations, otherwise the more restrictive column requirements of ACI 318-05 Section 21.4 are necessary. In wall lines where only a few elements are classified as columns, engineers have the option to ignore these members and classify them as "frame elements not proportioned to resist forces induced by earthquake motions" as long as they are detailed in accordance with ACI 318-05 Section 21.11.

The special reinforcing provisions for wall piers are not required when those piers under consideration are adequately braced by other wall segments of substantially larger stiffness. Such lateral bracing elements must have lateral stiffness to resist the lateral deflections of the story such that any resulting deflection will not affect the wall pier strength substantially. The ratio for the sum of relative stiffnesses between wall bracing elements to those of wall piers needs to be greater than or equal to six for the wall piers to be considered adequately braced. This was originally proposed in the Blue Book and has been in use in California and in the Uniform Building Code for a considerable amount of time, without incident. Some have questioned this relative stiffness and have suggested using a value of 12 based on a proposed code change in ACI 318 section 10.10.1 on the slenderness effect of compression members. At the present time, however, there are neither tests nor documented data known to the Seismology Committee to justify an increase from the original value of six.

Research done on thin tilt-up frame panels (similar to wall piers) has shown the benefits of close tie spacing in the hinge zone where flexural yielding initiates. Cyclic loading tests of full-scale tilt-up specimens provide insight into the behavior of wall piers that have various tie spacings. Four-inch tie spacing was effective in achieving ductility whereas eight-inch and twelve-inch spacings were not effective, allowing primary flexural steel to buckle (Dew, Sexsmith, and Weiler 2001).

Panel-to-Panel Connections

From its beginnings in the 1940s and into the 1970s, tilt-up wall panels were joined together with cast-in-place pilasters formed between adjacent panels. Horizontal panel reinforcing extended into the cast-in-place joint effectively making all the panels behave almost as monolithic concrete. Since the 1970s, panels have been left separated with dry panel joints, but building designs still require connections across these joints for diaphragm chords and drag struts. In addition, some engineers provide connections across the joints for overturning resistance.

Intermittent connections across the panel joints are vulnerable to force concentrations from concrete shrinkage strains and differential foundation settlement. The concrete panels are often lifted into place after only a week of curing with a substantial amount of concrete shrinkage potential remaining. Panels set on pad footings instead of continuous footings are more subject to differential settlement. This potential for relative movement between individual panels requires connections across these joints to be strong and well developed into the concrete to prevent brittle failure.

Chord connections and drag connections typically involve the use of reinforcing steel or rolled steel shape ledgers directly spliced at the panel joints. To accommodate the shrinkage strains that can occur between panels, the continuous chord members are often detached from the panel in the vicinity of the joint to allow for horizontal movement. In the case of reinforcing chord bars, the reinforcing is often wrapped or otherwise unbonded for a short distance from the panel joint. Where steel angle or channel ledgers are used, horizontally slotted bolt holes near the panel joints are provided. These methods of chord detachment allow the horizontal strains to better relieve themselves elastically across the unbonded chord distance and reduce the likelihood of brittle weld failure or concrete spalling at the connection.

Occasionally, overturning resistance of shear wall panels is supplemented with panel-to-panel embedment connections, effectively grouping adjacent panels together to behave as a whole. Often these connections are non-ductile with insufficient overstrength to prevent brittle concrete cone failure or weld rupture, and are not suitable in high seismic zones (Hofheins, Reaveley, and Pantelides 2002; Lemieux, Sexsmith, and Weiler 1998). Alternatively for overturning resistance, panels can be attached exclusively to the foundation for uplift forces allowing the panels to more freely move at their joints. It is recommended that any panel-to-panel connections for uplift resistance be used sparingly and only if they are well developed into the concrete with reinforcing welds designed to fully develop the reinforcing bars. ACI 318-05 Section 21.8 for "Special structural walls constructed using precast concrete" references Sections 21.13.2 and 21.13.3 which require panel-to-panel connections to yield within the steel elements or reinforcement, or otherwise be designed to develop at least 1½ times the connection yield strength. The SEAOC Seismology Committee recommends that panel-to-panel connections be designed with the Special Seismic Load Combinations per 2006 IBC 1605.4 in lieu of developing only 1½ times the connection yield strength.

Wall Connections to the Foundation

In concrete tilt-up construction, many regional areas have seen tilt-up panels not attached to the spread footing foundation that provides bearing support for the panels themselves. In these buildings, wall panels are vertically supported on the foundation, but lateral forces are resisted only by panel connections to the slab-on-grade. Typically these connections consist of reinforcing dowels from the wall panel and from slab-on-grade that lap within a narrow concrete pour-strip in the floor parallel to the wall panel.

ACI 318-05 Sections 15.8.3.2 and 16.5.1.3 allow precast concrete wall panels to forego the traditional footing connection requirements that cast-in-place walls have in Section 15.8.2.2. The exclusive connection to the slab is theoretically permissible provided that a rational load path is established to transfer the in-plane and out-of-plane forces through the slab-on-grade and to the supporting soil.

However, slab-sliding resistance is difficult to predict, especially where a plastic moisture/vapor retarder is provided below the slab. Also, it is desirable to mobilize the lateral sliding strength of the foundations. For these reasons, SEAOC Seismology Committee issued a "Position Statement" strongly recommending that designs in high seismically active areas include either a direct or indirect connection to the foundation (SEAOC 2000). An example of an indirect connection would be panel dowels tied into the slab-on-grade and additional footing dowels also tied into the slab-on-grade. It is very common to see tilt-up panels with no foundation connection, yet there has not been any reported damage associated with this in past earthquakes.

At heavily loaded shear walls, overturning may require an uplift connection from the wall panel directly to the foundation. ACI 318-05 Section 21.8 for "Special structural walls constructed using precast concrete" references Sections 21.13.2 and 21.13.3 which require panel-to-foundation connections to yield within the steel element or reinforcement, or otherwise be designed to develop at least 1½ times the connection yield strength.

New Thinking

Past performance of tilt-up buildings has been instrumental in making code revisions to wall anchorage and subdiaphragm requirements, but the process has been a series of incremental reactionary changes that have simply increased the anchorage force or added prescriptive detailing language. There is a belief among some engineers and

researchers that an entirely new approach may be needed in the seismic design of low-rise buildings with relatively rigid vertical elements and with flexible diaphragms, such as is common with tilt-up and masonry buildings today.

Much of the current building code seismic provisions were developed with classic lumped masses at the floor and roof assumed. The seismic response of the traditional lumped mass model is dominated by the stiffness of the vertically oriented lateral force-resisting system, and this assumption applies well to frame structures or tall shear wall structures with rigid heavy diaphragms.

Buildings with rigid shear wall systems supporting flexible diaphragms have several unique properties that make them behave substantially different than classic framing types. The short concrete or masonry shear walls have very little flexibility compared with the diaphragm, and thus the diaphragm period dominates the overall building seismic response (Fonseca, Wood, and Hawkins 1996). Analytical modeling and research has shown that this seismic response can be accurately modeled and predicted (Fonseca, Hawkins, and Wood 1999).

In contrast, the equivalent lateral force analysis in ASCE 7-05, as well as the 2006 IBC, estimates building seismic response as a direct function of building height and the lateral force-resisting system stiffness of vertical structural elements. In addition, current codes determine diaphragm and wall anchorage forces as a direct function of the ground motion only, or sometimes in consideration of the vertical lateral force-resisting system stiffness, without any consideration of the dynamic response of the diaphragm. This inaccuracy in current codes has led some to recommend entirely new code provisions for use in rigid shear wall / flexible diaphragm buildings. Freeman, Searer, and Gilmartin (2002) provide a rational approach that computes seismic response more accurately, based on diaphragm and wall out-of-plane periods. An approach similar to Freeman's has merit and should be considered as the basis for future code provisions. ASCE 7-05 Section 12.7.3 does provide general language for an alternative dynamic approach, however such a complex approach is not mandatory for typical tilt-up buildings and is outside of normal engineering practices.

Another unique aspect associated with rigid shear wall / flexible diaphragm buildings is that lack of diaphragm stiffness effectively isolates adjacent, parallel lines of shear resistance from each other. Unlike the case with non-flexible diaphragms, any softening or degradation of an overloaded shear wall that is beginning to fail cannot have its loads effectively transferred across the diaphragm to adjacent lines of shear resistance until significant wall translation has taken place. This amount of wall translation is likely to result in complete failure of the shear wall before any significant sharing of loads takes place among the other walls.

The implication is that the redundancy coefficient, ρ, governed by a redundancy weakness in only one line of resistance, is very critical to that line, but need not impact the design of other parallel lines of resistance due to the diaphragm isolation effect. In a theoretical sense, ρ should apply only to specific lines of resistance judged independently, but the value of ρ should be higher to reflect the extra critical nature of the isolated problem being unable to share the load through the flexible diaphragm to other parallel lines of resistance. However, some engineers argue that the purpose of ρ is not to apply a rational approach to strengthening non-redundant elements, but to penalize the entire building system as a discouragement to engineers.

Also related to the isolation effects of flexible diaphragms is the critical nature of struts or collectors in rigid shear wall / flexible diaphragm buildings. Again, because a failure in a collector line has limited ability to transfer loads through the flexible diaphragm to other parallel collector lines, these collectors are non-redundant. ASCE 7-05 Sections 12.10.2.1 and 12.4.3.2 have special load combination provisions for collectors that increase the design loads to address their critical nature. Rigid and semi-rigid diaphragms are more able to redistribute loads through the diaphragm should a collector failure occur, but current codes do not differentiate between the two in this regard.

Further research is justified for rigid shear wall / flexible diaphragm buildings, including tilt-up construction. A new approach to code provisions governing this building type would allow a more rational consideration of building response, including appropriate provisions for redundancy coefficients, collector designs, deformation compatibility issues, and diaphragm deflection limits.

In 2005, the Tilt-Up Concrete Association (TCA) formed a seismic design task group as a proactive measure to advance research on the unique dynamic behavior of this form of construction. Although the work of TCA is in its infancy, its collection of tilt-up construction experts and researchers has great potential in guiding future code revisions. Additionally, in 2007 the University of British Columbia began a research program partially funded by the Portland Cement Association to study the dynamic behavior of tilt-up buildings and their connections as influenced with flexible diaphragms.

As tilt-up construction continues to evolve architecturally and structurally, new questions arise for engineers to consider. The state of the art in tilt-up design has made much progress over the last several decades, but important questions continue to be at hand for additional research:

> Can a parallel set of seismic code provisions be developed for rigid shear wall / flexible diaphragm buildings that considers more accurately their seismic response?
>
> Is the selected method of distributing in-plane shears critical to shear wall performance?
>
> Can a simplified method of shear distribution achieve acceptable results?
>
> Are deformation limits for wall anchorage systems necessary, and if so, how should they be set?
>
> As wall anchorage is eliminated as the weak element of tilt-up structures, will the mode of failure simply transfer to another vulnerable portion of the system?

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