MODELING ISSUES FOR TALL REINFORCED CONCRETE CORE WALL BUILDINGS

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Abstract

Reinforced concrete walls are commonly used as the primary lateral-force-resisting system for tall buildings. As the tools for conducting nonlinear response history analysis have improved and with the advent of performance-based seismic design, reinforced concrete walls and core walls are often employed as the only lateral-force-resisting system. Proper modeling of the load versus deformation behavior of reinforced concrete walls is essential to accurately predict important response quantities. Given this critical need, an overview of modeling approaches appropriate to capture the lateral load responses of both slender and stout reinforced concrete walls is presented. Modeling of both flexural and shear responses is addressed, as well as the potential impact of coupled flexure-shear behavior. Model results are compared with experimental results to assess the ability of common modeling approaches to accurately predict both global and local experimental responses. Based on the findings, specific recommendations are made for general modeling issues, limiting material strains for combined bending and axial load, and shear backbone relations.

Introduction

Reinforced concrete (RC) structural walls are effective for resisting lateral loads imposed by wind or earthquakes. They provide substantial strength and stiffness as well as the deformation capacity needed to meet the demands of strong earthquake ground motions. As the tools for conducting nonlinear response history analysis has improved and the application of performance-based seismic design approaches have become common, use of reinforced concrete walls and core walls for lateral-force resistance along with a slab-column gravity frame have emerged as one of the preferred systems for tall buildings.

The lateral building strength is sometimes concentrated in relatively few walls distributed around the floor plate or within a central core wall to provide the lateral stiffness needed to limit the lateral deformations to acceptable levels. Although extensive research has been carried out to study the behavior of reinforced concrete walls and frame-wall systems, including development of very refined modeling approaches, use of relatively simple or course models is required for very tall buildings to reduce computer run times for the nonlinear response history analyses. Therefore, it is important to balance model simplicity with the ability to reliably predict the inelastic response both at the global and local levels under seismic loads to ensure that the analytical model captures the hysteretic wall behavior and the interaction between the wall and other structural members and the foundation reasonably well.

The most common modeling approach used for RC walls involves using a fiber beam-column element (e.g, PERFORM 3D, 2006) or the Multiple-Vertical-Line-Element Model (MVLEM), which is similar to a fiber model (Orakcal and Wallace, 2006). Use of either of these models allows for a fairly detailed...
description wall geometry, reinforcement, and materials, and accounts for important response features such as migration of the neutral axis along the wall cross-section during loading and unloading, interaction with the connecting components such as slabs and girders, both in the plane of the wall and perpendicular to the wall, as well as the influence of variation of axial load on wall flexural stiffness and strength. Important modeling parameters include the definition of the material properties for the longitudinal reinforcement, the core concrete enclosed by transverse reinforcement (i.e., confined concrete), and cover and web concrete (i.e., unconfined concrete). More complex model and material behavior also can be described that incorporates observed interaction between flexural/axial behavior and shear behavior (Massone et al, 2006; Orakcal et al., 2006); however, an uncoupled model, where flexure/axial behavior is independent of shear behavior, is commonly used for design.

Coupling or link beams exist due to the core wall configuration or they are needed to enhance the lateral stiffness of the building, and proper modeling of the load versus deformation behavior of reinforced concrete and coupling beams is essential to accurately predict important response quantities. Selection of appropriate flexural stiffness values for the coupling beams is particularly important, as use of one-half of the gross concrete inertia value generally produces higher shear stresses than are acceptable for design. Given this problem, it is common practice to reduce coupling beam stiffness to significantly lower values, on the order of $0.2I_g$ to $0.1I_g$, or less, to achieve an acceptable level of shear stress for design actions. At issue is whether a reduction of this magnitude is appropriate at service or design loads, since relatively large cracks may form, or at DBE or MCE, where a significant stiffness reduction would appear appropriate, and spalling of relatively large chunks of concrete could potentially pose a hazard.

It is common for the footprint of the building at lower levels to be larger than the tower footprint to accommodate parking needs. This abrupt change in geometry can have a significant impact on the distribution of lateral forces at the discontinuity, where loads are shared between the core wall and the parking level walls. Parking level walls are typically stout, i.e., the wall height-to-length or aspect ratio is low; therefore, selection of the stiffness values for flexure and shear are important and can substantially impact the distribution of lateral forces between the core wall and perimeter basement level walls. Since the floor slabs in the region of the discontinuity are required to transfer forces from the core wall to the perimeter walls, selection of appropriate slab stiffness values and design of slab reinforcement are important issues. Variation of slab and wall stiffness values is typically required to determine the potential range of design values to ensure proper design. For embedded basement levels, response history analysis is further complicated by the need to define the level at which the ground acceleration records are applied (Stewart., 2007).

Slab-column framing provides an efficient system to resist gravity loads, with limited forming and relatively low story heights, as well as providing an open, flexible floor plan. Since the slab-column frame resists only gravity loads, it is not included in the lateral load analysis. However, the ability of the slab-column gravity frame to maintain support for gravity loads for the lateral deformations imposed on the gravity system by the lateral-force resisting system must be checked. The primary objectives of this “deformation compatibility” check are to verify that slab-column punching failures will not occur for service-level and design-level earthquakes, as well as to assess the need to place slab shear reinforcement adjacent to the column to enhance slab shear strength. Design requirements for these checks are included in ACI 318-05 §21.11.5 (Building, 2005). Detailing of the slab-wall connection also is an important design consideration, as the rotation of the core wall can impose relatively large rotation demands on the slab at the slab-wall interface (Klemencic et al., 2006). During construction, construction and concrete placement of the core wall typically precedes the slab, requiring special attention to slab shear and moment transfer at the slab-wall interface.

As noted in the preceding paragraphs, design of tall buildings utilizing reinforced concrete walls is complicated by the uncertainty associated with a variety of issues. Although analytical modeling studies and experimental studies are appropriate for improving our understanding of some issues (e.g., coupling
beams), it is clear that other issues can only be answered by installation of sensors in actual buildings. Ideally, the sensors could be installed both during and after building construction to enable the broadest spectrum of data collection and follow-up analytical studies. Sensors to measure a wide variety of response quantities (acceleration, force/pressure, velocity, displacement, rotation, and strain) could be installed to collect critical data to improve our ability to model the dynamic responses of tall buildings. Data collection in for ambient, wind, low-level earthquakes, as well as the significant earthquakes would help improve our computer modeling capabilities, and ultimately the economy and safety of tall buildings.

The preceding paragraphs provide an overview of some of the important issues associated with analysis and design of tall reinforced concrete buildings. A more detailed discussion, including a review of relevant recent research and specific recommendations are presented in the following sections.

Wall Modeling

Orakcal and Wallace (2006) present the most comprehensive study available on the ability of current modeling approaches to capture the cyclic response of relatively slender reinforced concrete walls for combined bending and axial load. A MVLEM is employed in their study for planar, cyclic loading, which is conceptually the same as the fiber model approaches that are embedded in some commercially available computer programs (e.g., PERFORM 3D). Given the wall cross-section and the quantity of longitudinal and transverse reinforcement, the overall process presented by Orakcal and Wallace (2006) involves: (1) subdividing the wall cross section into unconfined concrete, confined concrete, and reinforcement fibers, (2) selecting appropriate material relations (3) subdividing the wall into a specified number of elements (components) over the wall height, (4) defining appropriate boundary conditions, and (5) imposing a prescribed load/displacement history. Results of their study are shown in Figure 1 for a test of a 12 ft tall wall with a 4 in. by 48 in. cross section subjected to constant axial load and reversed cyclic lateral displacements at the top of the wall. The test walls were approximately one-fourth scale models of prototype walls proportioned using the 1991 Uniform Building Code (Thomsen and Wallace, 2004).

\[
P = 0.07A_gf'c
\]

\[
u_{u,\text{max}} = 2.2\sqrt{f'c}\text{ psi}
\]

Figure 1 Test results for specimen RW2

It is noted that Orakcal and Wallace (2006) reduced the test data into lateral force versus deformation relations for flexure and shear. As well, spurious contributions from foundation rotation or slip between the test wall foundation and the strong floor were removed. Several important observations can be gleaned from the results. The effective linear stiffness to the yield point is very close to the \(0.5EI_g\) value commonly used for design and that the wall lateral load capacity computed using the nominal moment capacity at the wall critical section located at the wall base for as-tested material properties is slightly less
than the maximum lateral load achieved during the test (Thomsen and Wallace, 2004). Results from a recent shake table test of a full-scale rectangular wall suggests a lower effective stiffness, on the order of 0.2EI_y (Maffei, 2007); however, it is important to note that this test was conducted for relatively low axial load (> 0.05A_ff'f) and it is not apparent whether the test data were processed to separate responses into flexural and shear load versus deformation behavior or to account for any deformation contributions due to foundation flexibility. More importantly, a primary objective of the shake table test was to demonstrate that satisfactory lateral-load behavior could be achieved using approximately one-half the longitudinal reinforcement typically required by current codes in similar walls (e.g., UBC-97 or IBC-2003). Although a detailed assessment of the impact of the quantity of reinforcement used has not been conducted, it is noted that the tension reinforcement ratio for the shake table test
\[ \rho_{\text{boundary}} = \frac{8(0.31 \text{ in}^2)}{(8''(144''))} = 0.0022 \quad \text{(http://nees.ucsd.edu/7Story.html)} \]
which is less than one-half that used by Thomsen and Wallace (2004): \( \rho_{\text{boundary}} = \frac{8(0.11 \text{ in}^2)}{(4''(48''))} = 0.0046 \). Use of substantially lower longitudinal reinforcement would be expected to significant impact the effective stiffness to yield because the yield curvature is primarily a function of the wall length, i.e., \( \phi_y \approx (0.0025 \text{ to } 0.003) / l_y \) is commonly assumed (Wallace and Moehle, 1992). Therefore, for a given wall length, a reduction in the nominal (yield) moment strength by a factor of two will produce an approximately equal reduction in the effective stiffness (Figure 2). Given this result, the effective stiffness of 0.2EI_y reported for the shake table test is not inconsistent with the results given in Figure 1.

The results presented also indicate that cyclic material relations for concrete and reinforcing steel can be selected to produce overall load versus deformation responses which are generally consistent with test results for a wide range of responses (i.e., overall load versus roof displacement, plastic hinge rotation, and average strains). Orakcal and Wallace (2006) report that model and test results for first story displacements and rotations, where inelastic deformations dominate over elastic deformations, compare very favorably. Results for average wall strain over a nine inch gauge length at the base of the wall (Figure 3) reveal that tensile strains are well represented with the mode; however, model compressive strains substantially underestimate the peak compressive strains measured during the test. In general, for the relatively slender wall tests \( \left( k_w l_w = M_u / (l_w) = 3 \right) \), peak measured compressive strains were about twice the model predicted strains. Therefore, until more information is available (test and model), limits placed on maximum compressive strains derived from model predictions should be doubled to compare

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**Figure 2** Wall effective stiffness

**Figure 3** Wall average strain at critical section
with acceptance values for compressive strain (or the limit should be halved to compare with model strain values).

The results presented in Figure 1 and 3 include nonlinear flexural behavior; however, linear shear behavior is assumed, i.e., that is flexural behavior and shear behavior are uncoupled. It is apparent from the experimental results presented in Figure 4 for first story deformation that significant inelastic shear deformations initiate at the same instance as inelastic flexural deformations, that is, the flexural and shear responses appear coupled. It is noted that currently available computer programs do not account for the apparent coupling of flexure and shear behavior observed in Figure 4. The analysis results presented in Figure 4 are for a coupled model for monotonic material behavior (Massone et al., 2006) and reveal that a coupled model can reproduce observed test results reasonably well. However, models that account for coupled flexure-shear behavior for cyclic loading are not yet available in commercially available computer programs as development of coupled cyclic material models remains a significant research challenge.

Figure 4 Load displacement relations: (a) Flexure, (b) Shear

Determination of an appropriate model for nonlinear response analysis requires subdividing both the cross section into concrete and steel fibers, and subdividing the overall wall into elements of appropriate heights to capture salient responses. Results presented in Figure 5 reveal that a lateral load versus lateral top displacement relations is insensitive to the number of material fibers and number of elements used, that is, mesh and element refinements do not markedly improve the response prediction (Orakcal et al., 2004). This result is encouraging in that a coarse mesh can be used to assess drift responses for tall buildings, leading to reduced computer run times. However, results presented in Figure 6 present an important corollary, that use of a coarse mesh is likely to underestimate the peak strains for the material fibers (Orakcal et al., 2004). The peak model compressive strains using 8 elements and 8 fibers are approximately 30% less than the strains obtained using 17 elements and 22 fibers. Therefore, use of a fiber model with a relatively coarse mesh, although able to accurately represent the overall wall lateral load versus top displacement, may substantially under predict the maximum compressive strain at the wall critical section. For the results presented for the coarse mesh, measured peak compressive strains exceed model results by a factor of more than 2.5 (30% due to the mesh, and a factor of two from Figure 3); therefore, an acceptance criterion with a limiting peak compressive strain of 1% implies actual peak compressive strains might be closer to 2.5%. Given the information presented, acceptance criteria for wall strains should carefully consider the model attributes to establish an appropriate limit for peak concrete compression strain.
Depending on the selection of the material relations and model attributes, model estimates of local deformations may be impacted by localization of inelastic deformations. For example, consider a model with a relatively large number of elements at the wall critical section and bilinear material model with no post-yield stiffness are used to model the test wall RW2 discussed earlier (Figures 1 and 3). The yield displacement is approximately 0.5 in.; therefore, to achieve a peak top lateral displacement of 3.0 inches as observed in the test, the plastic rotation over a plastic hinge length of one-half the wall length ($l_w/2 = 24$ in.) is $	heta_p = (3.0 - 0.5)/(150 - 24/2) = 0.018$.

However, if an element height of 4 in. is used, the associated curvature would be $\phi = 0.0018/4" = 0.0045/in.$ and the peak wall compressive strain could be estimated as approximately $\varepsilon_{c,\text{max}} = \left(\phi + \phi_p\right)\left(c \approx 0.15/l_w\right) = 0.003/l_w + 0.00075/in.\left(0.15\right)\left(48^\circ\right) = 0.033$, whereas the peak measured concrete strain in the test reached only 0.006. The strains predicted with the model are unrealistically large due to the short element height combined with the lack of post-yield stiffness, which limits yielding to only one element (inclusion of some post-yield stiffness would allow the base moment

to increase and eventually spread yielding to adjacent elements above the element at the base). Therefore, in this example, the element at the wall base should be selected to be approximately equal to the expected plastic hinge length, or approximately ($l_w/2 = 24$ in.), which would produce an estimate of the peak compressive strain of $\varepsilon_{c,\text{max}} = \left(\phi + \phi_p\right)\left(c \approx 0.15/l_w\right) = 0.003/l_w + 0.00075/in.\left(0.15\right)\left(48^\circ\right) = 0.006$, which is consistent with test results. In general, the element height at the wall critical section should be approximately equal to the expected plastic hinge length and modest post yield stiffness should be used to help avoid problems associated with localization of inelastic deformations. Use of a modest strain hardening slope of 3 to 5% on the reinforcement stress versus strain relation is a good means to accomplish this goal.

Massone et al. (2006) present a modeling approach that incorporates flexure-shear interaction for monotonic loading. Details of the modeling approach are beyond the scope of this paper; however, results for the slender walls discussed earlier and shown in Figure 4 indicate that the model captures the observed interaction between flexure and shear (shear yielding not observed in uncoupled models). For the coupled analysis, larger concrete compressive strains results relative to an uncoupled analysis. A rigorous analysis
is needed to assess the magnitude of the impact of the coupled analysis on peak compressive strains; however, the preliminary results suggest that the coupled analysis is responsible for some of the discrepancy between the wall strains predicted with an uncoupled model and the test results.

**Coupling Beam**

Current code provisions for diagonally reinforced coupling or link beams produce designs with substantial congestion of reinforcement (Figure 7(a)) making them difficult and expensive to construct. New detailing provisions will be introduced in ACI 318-08 Chapter 21 to address this issue. The new provisions allow two options, one similar to prior ACI codes where transverse reinforcement is placed around the diagonal bars (Figure 7(b)) and modest transverse reinforcement is used around the entire beam section, and an alternative where a larger quantity of transverse reinforcement is provided around the entire cross section is confined (Figure 7(c)).

A test program is underway at UCLA to assess the response of link beams using the two options identified in ACI 318-08. The test specimens are one-half scale and the test geometries and reinforcement were selected to be representative of common conditions for residential \((l/h=36''/15''=2.4)\) and office \((l/h=60''/18''=3.33)\) construction. Diagonal reinforcement consists of a six-bar arrangement of #7 US Grade 60 reinforcing bars with expected maximum shear stresses of approximately \(\nu_{u,\text{max}} = 6\sqrt{f_c} \text{ psi}\) and \(\nu_{u,\text{max}} = 10\sqrt{f_c} \text{ psi}\) for the link beams with span to total depth ratios of 3.33 and 2.4, respectively. Reinfocing details for the two specimens groups of specimens are shown in Figure 8. To date, the test results are only available for the two beams depicted in Figure 8(a), with span to total depth ratios of 3.33. Although only preliminary test results are available, important preliminary findings are summarized in the following paragraphs.

![Figure 7(a) Coupling beam with reinforcement congestion](image)

![Figure 7 Link beam details: (b) ACI 318-05, (c) ACI 318-08 proposed](image)

The shear force versus link beam rotation relations for the two tests are presented in Figure 9(a) and reveal that very similar force-deformation responses were obtained using the different detailing schemes. Rotation levels exceeding 8% were achieved for both tests with virtually no strength degradation. The peak link beam shear stress reached approximately \(\nu_{u,\text{max}} = 6\sqrt{f_c} \text{ psi}\) (Figure 9(b)) and the force-deformation loops for each beam were nearly identical, indicating that the detailing arrangement given in Figure 8(b) was as effective as that for Figure 8(a) for this beam configuration. Figure 9(b) plots the effective secant stiffness as a fraction of
concrete gross section stiffness for each test specimen and reveals the effective stiffness was approximately 0.25 to 0.40$I_g$ for initial loading and degraded to 0.1$I_g$ when the peak shear stress was reached at 1% rotation and dropped to 0.05$I_g$ and 0.03$I_g$ for 2% and 4% rotations, respectively. The relatively low initial stiffness may have been influenced by minor initial cracking that existed in the test specimens. A more thorough assessment of appropriate values for the effective moment of inertia will be conducted at the completion of the test program.

Link beam stiffness is often reduced during the design phase to reduce link beam shear stresses to code-acceptable levels (i.e., $v_{n,max}=10\sqrt{f_c}$ psi) giving rise to concerns that excessive crack widths may be required to achieve the assumed stiffness values. However, the preliminary test results indicate only hairline to 1/64” diagonal crack widths and flexural crack widths of 1/8” to 3/16” for lateral drift levels of...
3% to 4% at peak displacement. Residual crack widths at 4% drift were approximately 1/32” for flexural cracks and up to 1/64” for diagonal cracks. Photos of the specimens at several drift ratios are provided in Figure 10. It is noted that larger crack widths are likely for the beams with shorter span to total depth of 2.4 given the higher shear stress anticipated; however, preliminary results indicate that use of a reduced stiffness for link beams is unlikely to require excessive cracking, either at service load levels or for rotation limits used for the DBE.

As noted earlier, use of post-tensioned slab-column frames with reinforced concrete core walls has become relatively common for tall buildings. The existence of the slab post-tensioning reinforcement will restrain the axial growth of the beam (which was unrestrained in the tests described above) and impact the test results. Therefore, link beam tests are planned to study the impact of the post-tensioning reinforcement on link beam performance. The axial compression imposed by the post-tensioning would be expected to increase link beam strength and stiffness, and potentially reduce deformation capacity. Recent tests on lightly-reinforced, three-quarter scale piers, indicated that modest axial load of $0.05A_g f_c'$ (200 psi) increase pier shear strength by approximately 50% over the zero axial load strength. The upcoming tests will shed important light on these issues.

**Modeling of shear-dominated wall segments**

Relatively limited information exists on appropriate modeling approaches for reinforced concrete wall segments with shear-dominant behavior. A review tests results by Sozen and Moehle (1993) indicated that use of a bilinear force versus deformation relation defined by a cracking stress and a post-yield slope reasonably represented the limited test results available. Recent tests and modeling studies, although primarily conducted on lightly-reinforced wall piers, provide significant data to make new recommendations as summarized in the following paragraphs.

Based on a review of test results for wall piers, a force versus deformation relation based on the following points was determined:

![Figure 10 Damage photos of Beam #7: l/h = 3.33 specimens](image-url)
As well, for lightly reinforced wall piers, shear strength degradation occurred at a deformation of approximately 0.0075 times the specimen height \((0.0075h_w)\), as described in FEMA 356 (2000). Residual strength for the piers tested by Massone (2006) was low given that the piers were lightly-reinforced and generally contained poor details (Massone, 2006). Comparisons of the proposed backbone curve with experimental results for several tests are given below in Figure 11.

The results presented in Figure 11 indicate that the relation defined by Equation (1.1) provides a reasonable fit to precracked behavior and the nominal capacity and the yield deformation for lightly-reinforced tests with zero axial load. For the wall pier with modest axial load \((P = 0.05A_gf'_c)\), the ratio of the peak load in the test to the nominal capacity using Equation (1.1) is roughly 1.5 and deformation associated with the yield point is less than 0.004, indicating that the axial load produces a noticeably stiffer response. However, test data for wall piers with axial load are very limited; therefore, a more detailed assessment of the yield deformation for wall piers with axial load is not possible. Until more data are available, the relation described by Equation (1.1) is recommended.

The modeling approach for coupled flexure-shear responses was used to compare model results with test results for several walls. Results are presented in Figure 18 and described in more detail by Massone et al. (2006) and Massone (2006).

As observed in Figure 12(a), a very good correlation is obtained between test results and results of the proposed coupled shear-flexure model for Specimen 74 \((M/(Vl)=1.0)\). Since the design flexural and shear capacities of the specimen are close, to consider the possibility that the response of the specimen is governed by nonlinear flexural deformations (i.e., the specimen does not experience significant nonlinear shear deformations), Figure 12(a) also includes an analytical flexural response prediction (with shear deformations not considered) obtained using a fiber model. The same geometric discretization and
material constitutive models used in the coupled model were adopted for the fiber model, with the distinction that the panel elements (strips) of the coupled model were replaced with uniaxial (fiber) elements. Figure 12(a) illustrates that although the flexural (fiber) model provides a ballpark estimation of the wall lateral load capacity, (predicted lateral load capacities approximately 750 kN and 980 kN, respectively for the coupled and flexural models), the load-displacement response obtained by the flexural model is significantly different than the measured response and the coupled model response. After a lateral load of 450 kN, significant lateral stiffness degradation is observed in both the test results and results of the coupled model, but not in with the flexural model.

This result demonstrates how the proposed model, which couples shear and flexural responses, is able to simulate observed responses with substantially greater accuracy than a flexural model, particularly for wall specimens where the nominal shear and flexural capacities are nearly equal. The correlation between results of the coupled model and test results for Specimen 10 ($M/V_{lw} = 0.69$) is similar to that of Specimen 74 ($M/V_{lw} = 1.0$) (Hidalgo, 2002). Hiwosawa (1975)

This result demonstrates how the proposed model, which couples shear and flexural responses, is able to simulate observed responses with substantially greater accuracy than a flexural model, particularly for wall specimens where the nominal shear and flexural capacities are nearly equal. The correlation between results of the coupled model and test results for Specimen 10 ($M/V_{lw} = 0.69$) is similar to that of Specimen 74 (Figure 12(b)). The model provides a good prediction of the lateral load capacity and lateral stiffness of the wall specimen for most of the top displacement history, although the wall specimen reaches its peak lateral load capacity at a smaller top displacement than that predicted by the model. The sudden lateral load reductions observed in the model response are due to sequential cracking of concrete,
whereas such behavior is not observed in test results. Refining the model discretization (increasing the number of strips used along the length of the wall) would invoke a more gradual and continuous shape for the analytical load-displacement response. Model predictions are not as good for walls with (\(M/(Vl) > 0.5\)) because model assumptions are not appropriate for such low aspect ratio walls where the end restraint significantly impacts the wall strain distribution. Massone (2006) indicate that use of a nonlinear horizontal strain profile can be used to improve response predictions for such walls.

**Slab-column frames**

Detailed modeling information is presented in ASCE 41 Supplement #1 (Elwood et al., 2007) as well as recent papers (Kang and Wallace, 2005, 2006, Kang et al. 2006); therefore, only a brief overview summary of critical issues is included here.

In recent research (Kang and Wallace, 2005, 2006), use of an effective slab width model with coefficients for the elastic, uncracked effective with (\(\alpha\)) and for cracking (\(\beta\)) are recommended for both reinforced concrete and post-tensioned concrete construction. The potential for punching shear failure at the slab-column connection is modeled using a rigid connection element (Kang et al. (2006); Elwood et al., 2007) with the connection stress limit defined using the eccentric shear stress model of ACI 318. However, given the detailed calculations required to check slab-column punching requirements, the relatively low cost of providing slab shear reinforcement (according to some engineers), as well as the improved performance that use of shear reinforcement provides (Kang and Wallace, 2005), it is fairly common to provide slab shear reinforcement at all slab-column connections without calculation (to satisfy the requirements of ACI 318-05 §21.11.5 where shear reinforcement is required).

Although it is relatively easy process to determine slab effective widths using the approach described in the preceding paragraph for slab-column frames and additional work has been conducted for slab-wall coupling (Wallace et al., 1990; Qadeer and Smith, 1969; Paulay and Taylor, 1981), current practice does not consider the impact of the coupling between the slab-column frame used as the primary gravity force system and the lateral system (e.g., reinforced concrete walls). This approach is based on common practice for seismic design, where the strength and stiffness contribution of the gravity system are neglected. However, as adjustments are made to refine the effective stiffness used for walls and columns (Elwood et al., 2007), as well as the impact of soil-foundation flexibility, this approach may need to be revisited for tall buildings. Parametric studies of several typical core wall configurations for various building heights would be helpful in assessing the influence of slab coupling on overall system behavior (e.g., drift, inelastic demands).

**Instrumentation for seismic monitoring**

The boom in tall building construction also provides a unique opportunity to employ monitoring equipment to measure structural responses for a variety of conditions (ambient, high-level wind, and earthquake). Ideally, a broad spectrum of sensor types capably of measuring floor accelerations, wind pressures, average concrete strains, rebar strains, and rotations should be employed. In addition to a broad spectrum of sensors, key attributes of a robust monitoring system include: rapid deployment, energy efficiency, event detection, robust analog-to-digital conversion, local storage, redundant time synchronization, multi-hop wireless data transport, and remote sensor and network health monitoring. Recent developments in all of these areas reveal that robust structural health monitoring is likely to emerge over the next decade. Therefore, careful consideration should be given to sensors in existing and planned buildings.

Given the complexity and geometry of tall buildings, laboratory studies, which are hindered by scale, materials, and appropriate boundary conditions, are unlikely to provide definitive results for a variety of issues. For a given instrumented building, the details of the embedded network design should be model-
driven, i.e., sensor types and locations determined based on response quantities obtained from 3D dynamic finite element models (FEM) subjected to a suite of site-specific ground motions. For example, in moment frames, response quantities of interest might be interstory displacements at several floors (where maximum values are expected) along with base and roof accelerations. In cases where a novel systems or materials are employed (e.g., high-performance concrete, headed reinforcement, unbonded braces), additional instrumentation could be used to measure very specific response quantities (e.g., headed bar strain, axial deformations, etc). In a concrete core wall system, response quantities of interest might be average core wall concrete strains within the plastic hinge (yielding) region and rotations imposed on coupling beams (or slab-wall connections). Other modeling and design issues could also be targeted, such as so-called podium effects and appropriate ground motion building inputs at subterranean levels. Given the uncertainty associated with the response of structural systems to earthquake ground motions, a probabilistic distribution of response quantities of interest (e.g., interstory displacements, coupling beam deformations) should be determined for the structural model subjected to the suite of ground motions and the sensor layout determined to target specific regions versus a single response quantity.

The City of Los Angeles requires building instrumentation (accelerometers) be installed at the base, mid-level, and roof to obtain a building permit for all buildings over ten stories as well as for buildings over 6 stories with an aggregate floor area exceeding 60,000 ft² (LA Building Code §1635, 2002). The owner is required to maintain the instrumentation in working order; the City of LA has an extensive program for monitoring the equipment currently installed in approximately 400 buildings. Currently, data collected by the required accelerometers are not archived and are not readily available for use either for rapid post-event assessment or by researchers to improve our ability to model buildings. Clearly, there are buildings where the measurement of interstory drift (moment frame) or average concrete strain (base of a shear wall system) might produce more useful and meaningful data than acceleration data alone. The instrumentation requirements for the City of Los Angeles are currently being updated to allow a more diverse array of sensors to provide more meaningful response quantities for very tall buildings.

Conclusions

An overview of some important issues associated with analysis and design of tall reinforced concrete wall buildings was presented. Based on this review, the following observations and conclusions are noted.

Existing commercially available computer programs that incorporate fiber models (or similar models) are capable of reproducing lateral-load versus top displacement relations measured from moderate-scale, relatively slender walls subjected to constant axial load and cyclic lateral displacements. Based on a review of test data, an effective stiffness of \(0.5I_g\) is appropriate for rectangular walls unless very light boundary reinforcement is used. Model element heights used within the potential plastic hinge region should be selected to be approximately equal to the anticipated plastic hinge length and a modest reinforcement strain hardening ratio of 3 to 5% should be used to avoid potential problems associated with concentration of inelastic deformations within a single element of short height. Sufficient elements and fibers should be used to ensure the strain distribution along the cross section is adequately represented; however, even with these steps, current models underestimate the peak compressive strains measured in a limited number of tests by a factor of about two. Coupling between nonlinear flexural and shear deformations appears to be one factor that could explain this observed discrepancy.

Interaction between flexure and shear can substantially impact the load-deformation response of stout walls which are common at lower levels of tall buildings due to podium/parking levels and foundation walls. For stout walls, it is important to consider the impact of concrete cracking on the lateral stiffness. In most existing commercially available computer programs, nonlinear shear behavior is modeled independently from axial-flexural behavior. A modified backbone relation which defines cracking and yielding points is recommended for stout walls with low axial load. The proposed relation captures the
load-deformation response of lightly-reinforced wall segments with very low axial load reasonably well; however, for even modest axial load levels \( P = 0.05 A_f' \), the proposed model underestimates the peak strength and overestimates the yield displacement based on very limited test data.

New recommendations for modeling slab – column frame stiffness and punching failures has been incorporated into ASCE 41 Supplement #1 to assess lateral load response. As design documents incorporate more realistic estimates of component stiffness, modeling of the complete lateral and gravity systems for nonlinear response history analysis may be appropriate provided computer run times are not excessive (and continue to improve dramatically).

Given the complex issues that arise for tall buildings and the limited benefit of laboratory testing for such large structures, an aggressive program to incorporate building instrumentation is needed. A program is underway in Los Angeles to address this need.

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