Boundary Detailing of Coupled Core Wall System
Wall Piers

by

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Boundary Detailing of Coupled Core Wall System Wall Piers

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Abstract: The seismic provisions of ACI 318-08 permit two different procedures for evaluating the need for “special” boundary elements in structural walls. The first method is based on an approximation of the maximum fiber concrete compressive stresses calculated using wall gross section properties and an assumed linear distribution of stresses over the depth. The second method places a limit on the compression zone depth of the wall. These methods are based on studies of uncoupled continuous walls, and their applicability to coupled walls is unclear. A parametric study consisting of eight coupled core wall buildings with varying geometric dimensions is presented. The states of concrete stress, strain, and compression zone depth in the wall piers at code level forces are investigated. Based on evaluation of the two procedures permitted by ACI 318-08, it is concluded that neither procedure is appropriate for wall piers in coupled core wall systems. Cross-sectional fiber analysis is a more effective evaluation tool.

Key words: coupling beams, coupled core wall systems, reinforcement detailing, special boundary elements, structural walls, wall piers.

1. INTRODUCTION
The total normal stresses acting on the cross-section of a wall pier in a coupled core wall (CCW) system subjected to gravity and seismic loads are a combination of normal stresses resulting from (1) axial forces due to gravity loads, (2) axial forces resulting from frame action due to the coupling effect, and (3) overturning moments due to bending. The stresses resulting from frame action and overturning moments are present only when lateral loads are applied to the wall system.

In order to maintain sufficient axial and flexural capacity of the wall piers, longitudinal reinforcing steel must be transversely restrained to prevent buckling. The surrounding concrete can provide the required restraint to buckling provided that spalling of the concrete cover, concrete strength degradation, and damage due to shearing effects are mitigated. Where sufficient protection of the longitudinal reinforcement is not expected to be provided by surrounding concrete, transverse steel is provided to confine the core of the concrete in which the reinforcing steel is located. This core of confined concrete is referred to as a boundary element. See Figure 1 for a representation of special boundary elements on wall piers. In cases where large compressive strains are expected in the boundary zones of wall piers, heavily-tied boundary elements, referred to as “special” boundary elements in ACI 318 (2008), are required.

The seismic provisions of ACI 318 Building Code Requirements for Structural Concrete (2008) offer two methods for determining the need for special boundary elements. The first method (ACI 318 2008 §21.9.6.3) considers concrete stress, and permits expected maximum concrete compressive stresses to be computed assuming a linear stress distribution over the depth of the section using gross section properties.

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When the maximum concrete stress computed at factored loads, including earthquake effects, exceeds 20% of the specified concrete strength, special boundary elements are required. The second method (ACI 318 2008 §21.8.6.2) considers the location of the neutral axis of a section when subjected to factored loads. When the length of the compression zone exceeds a critical value, special boundary elements are required. ACI 318 (2008) §21.9.6.2 requires special boundary elements under the following condition:

$$c \geq \frac{l_w}{600 \left( \frac{\delta_u}{h_w} \right)}$$

In Eqn 1, $c$ is the depth of the compression zone, $l_w$ is the length of the wall being considered, $\delta_u$ is the design lateral displacement of the wall (or wall segment) being considered, and $h_w$ is the height of the wall (or wall segment) being considered; the ratio $\delta_u/h_w$ may not be taken less than 0.007. Eqn 1 is based on a displacement-based design approach, and assumes that special boundary elements are required when the strain at the extreme compression fiber of a wall section exceeds a critical value. It is important to realize that this provision was adopted for cases in which the walls are continuous, a single flexure-critical section can be assumed, and the cross-sectional properties of the wall or wall segment are constant. Structural concrete components are capable of resisting extreme fiber compression strains of 0.004 without significant strength degradation or spalling. Based on this, Wallace (1994) and Wight et al. (1996) have shown that special boundary elements are not required for a large class of reinforced concrete buildings utilizing cantilevered walls.

However, the magnitude of the normal strains in structural walls resisting a combination of gravity and lateral loads is sensitive to the magnitude of axial load in the structural walls. Considering the relatively large axial loads in the wall piers of CCW systems, maximum concrete normal strains are expected to be larger in wall piers of CCW systems than those in cantilevered walls. Furthermore, due to overturning moments induced through coupling beam shear transfer, the wall piers cannot be assumed to be continuous and have a single flexure-critical section. Furthermore, wall pier reinforcement in CCW systems is commonly varied over the height of the building. With this in mind, a parametric study was conducted to determine if there exists a family of CCW systems where maximum concrete normal strains in the wall piers do not exceed a critical value assumed to be equal to 0.004, and therefore would not require special boundary elements. It should be noted that this study is not intended to demonstrate that “ordinary” boundary elements are not required. A minimum amount of transverse boundary element steel, as required by ACI 318-08 §21.9.6.5 should be provided where special boundary elements are found not to be required.

2. RESEARCH SIGNIFICANCE

The lateral force resisting mechanisms in the wall piers of coupled core wall (CCW) systems involves, in part, generation of significantly large axial forces in the wall piers resulting from an accumulation of shear in the coupling beams over the height of the structure. The evaluation of the current ACI 318 (2008) provisions for
evaluating the need for special boundary elements in CCW wall piers presented in this paper illustrates the need for design provisions specific to wall piers subjected to large axial loads as is common in wall piers in CCW systems.

3. OBJECTIVE OF PARAMETRIC STUDY

The objective of the parametric study presented in this paper is to investigate the need for special boundary elements in the wall piers of CCW systems. Where special boundary elements are not required, steel congestion can be minimized thereby minimizing construction costs as well as construction difficulties. The applicability of the ACI 318-08 procedures are evaluated using cross-sectional fiber analysis as a tool for evaluating the extreme fiber concrete compressive stresses and strains.

4. PARAMETERS OF STUDY

Eight different configurations of buildings utilizing the CCW system were selected based on building footprints considered to be practical. Unencased built-up steel I-shape coupling beams (SCB) are used in all the buildings as this type of coupling beam will generate larger axial forces in the wall piers due to the superior stiffness of the SCB’s over reinforced concrete coupling beams as shown in previous studies (Shahrooz et al. 1993; Harries et al. 1997; Fortney et al. 2004).

All 20-story structures were designed using NEHRP (2000). The structures have 2.74m (108in) floor-to-floor heights, 178mm (7in) thick post-tensioned floor slabs, 610mm (24in) thick wall piers with 35MPa (5,000psi) concrete strength, and 248MPa (36ksi) steel for the beams. The structures are assumed to be located in San Francisco on a Class C (soft rock) site where $S_s = 1.5g$ and $S_1 = 0.65g$. The site falls into Seismic Use Group 1 and Seismic Design Category D. Floor gravity loads are assumed to be mechanical, electrical, plumbing: 0.24kPa (5psf), partitions: 0.48kPa (10psf), columns: 0.24kPa (5psf) for dead loads, and 2.4kN/m$^2$ (50psf) for live loads. Cladding dead load along the perimeter of the buildings is assumed to be 4.4kN/m (300plf).

Two different configurations of CCW systems were investigated: (1) one-cell cores consisting of two C-shape walls (Type A), and (2) two-cell cores consisting of two C-shape walls with a central I-Shape (Type B). Four buildings of each type were investigated. Refer to Figure 2 and Table 1 for the plan dimensions of the eight different structures. The core walls are the primary lateral-force-resisting system.

5. ANALYSIS AND CALCULATION OF DESIGN FORCES

For the structures considered in this study, NEHRP (2000) permits the use of the equivalent lateral force (ELF) procedure for determining the design lateral loads. To facilitate the analyses, the eight structures were modeled and analyzed using ETABS Nonlinear (Computers and Structures, Inc.). Appropriate load combinations, per NEHRP (2000), were considered. The earthquake load cases included bi-directional effects and accidental torsion. Dead and live gravity loads, as discussed previously, were also incorporated in the analyses and designs. Two earthquake load cases were considered. See Figure 3 for the assumed loading directions. Load case one (LC1) considers an earthquake acting West-to-East where 100% of the design basis earthquake (DBE) loads are applied in the West-East direction with 30% of the DBE loads applied in the South-North direction. Load case two (LC2) considers the DBE acting South-North where 100% of
the DBE loads are applied in the South-North direction with 30% of the DBE loads acting in the West-East direction. In both cases, accidental torsion is considered and is applied so as to increase the impact of bi-directional effects.

The individual walls were modeled as shell elements, and the three legs of each C or I-Shape were assigned as ETABS “piers.” Doing so allows the output to provide forces acting on each wall pier rather than on the individual shell elements that define the legs of the wall piers. To account for cracked sections in the elastic analyses, 0.35EIg was used for the base wall pier and 0.7EIg was used for all wall piers above the base wall pier. The reduced concrete moduli are in accordance with those recommended by ACI 318 (2008) §10.10.4.1. The coupling beams were modeled as line elements. Shell elements were used to model the floor plates, which were assumed to be rigid diaphragms. The corner columns were assigned infinitesimal cross-sectional areas, and were assigned pinned-pinned boundary conditions to ensure no contribution to lateral stiffness. These corner “dummy” columns are needed as part of the ETABS modeling requirements.

The superimposed gravity loads were taken into account by assigning ETABS “area masses” to the floor plates. Additionally, cladding weight was accounted for by assigning ETABS “line masses” to the perimeter edges of the floor plates. These additional loads ensure contribution of the superimposed gravity dead loads to the total dynamic weight of the structure.

### Table 1. Plan dimensions of structures (refer to Figure 2)

<table>
<thead>
<tr>
<th>Struct #</th>
<th>Type</th>
<th>X (m/ft)</th>
<th>Y (m/ft)</th>
<th>CX (m/ft)</th>
<th>CY (m/ft)</th>
<th>WL (m/ft)</th>
<th>FL (m/ft)</th>
<th>WLI (m/ft)</th>
<th>BL (m/ft)</th>
</tr>
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<tbody>
<tr>
<td>01</td>
<td>A</td>
<td>24.4 (80)</td>
<td>24.4 (80)</td>
<td>9.1 (30)</td>
<td>6.0 (20)</td>
<td>3.7 (12)</td>
<td>4.7 (15.5)</td>
<td>n/a</td>
<td>1.8 (6)</td>
</tr>
<tr>
<td>02</td>
<td>B</td>
<td>30.5 (100)</td>
<td>24.4 (80)</td>
<td>13.4 (44)</td>
<td>6.7 (22)</td>
<td>2.6 (8.5)</td>
<td>5.5 (18)</td>
<td>4.6 (15)</td>
<td>1.8 (6)</td>
</tr>
<tr>
<td>03</td>
<td>B</td>
<td>42.7 (140)</td>
<td>30.5 (100)</td>
<td>21.9 (72)</td>
<td>7.9 (26)</td>
<td>4.6 (15)</td>
<td>7.9 (26)</td>
<td>9.2 (30)</td>
<td>1.8 (6)</td>
</tr>
<tr>
<td>04</td>
<td>A</td>
<td>30.5 (100)</td>
<td>30.5 (100)</td>
<td>9.1 (30)</td>
<td>6.0 (20)</td>
<td>3.7 (12)</td>
<td>4.7 (15.5)</td>
<td>n/a</td>
<td>1.8 (6)</td>
</tr>
<tr>
<td>05</td>
<td>B</td>
<td>42.7 (140)</td>
<td>30.5 (100)</td>
<td>16.5 (54)</td>
<td>9.1 (30)</td>
<td>4.0 (13)</td>
<td>7.9 (26)</td>
<td>4.9 (16)</td>
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<tr>
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<td>B</td>
<td>30.5 (100)</td>
<td>24.4 (80)</td>
<td>14.6 (48)</td>
<td>6.7 (22)</td>
<td>3.1 (10)</td>
<td>5.5 (18)</td>
<td>4.9 (16)</td>
<td>1.8 (6)</td>
</tr>
<tr>
<td>07</td>
<td>A</td>
<td>24.4 (80)</td>
<td>24.4 (80)</td>
<td>6.7 (22)</td>
<td>4.9 (16)</td>
<td>2.4 (8)</td>
<td>3.7 (12)</td>
<td>n/a</td>
<td>1.8 (6)</td>
</tr>
<tr>
<td>08</td>
<td>A</td>
<td>30.5 (100)</td>
<td>30.5 (100)</td>
<td>6.7 (22)</td>
<td>4.9 (16)</td>
<td>2.4 (8)</td>
<td>3.7 (12)</td>
<td>n/a</td>
<td>1.8 (6)</td>
</tr>
</tbody>
</table>

### Figure 3. Locations of interest for stress calculations and reported stresses and strains for (a) tension wall, (b) compression wall, (c) I-shaped wall, and nomenclature used in equations

LC1 = Earthquake in coupled direction including directional effects and *accidental torsion.
LC2 = Earthquake in uncoupled direction including directional effects and *accidental torsion.
*Accidental torsion amplifies directional effect.
The resulting dynamic weights, seismic response coefficient, base shears, redundancy factors, wall overstrength factors, and distribution of forces are reported in Fortney (2005). Refer to Harries and McNiece (2005) and Fortney et al. (2008) for additional discussions related to wall overstrength factors in CCW systems.

6. COUPLING BEAM DESIGN
The steel coupling beams were designed similarly to a steel link beam in an eccentrically-braced frame (EBF). Properly designed steel links have been shown to exhibit excellent energy-absorbing characteristics as shown in previous studies (Roeder et al. 1978; Malley and Popov 1983; Kasai and Popov 1986). The link beam in an EBF is the primary element for dissipating energy, and is similar to the behavior of a coupling beam in a coupled core wall (CCW) system. The design and detailing requirements are provided in the Seismic Provisions for Structural Steel Buildings (Third Edition).

Beam sizes were varied over the height of the building in an effort to minimize the wall overstrength factors, thereby reducing the design forces for the wall piers. For all the structures, the maximum design beam shears for levels 1-4, 5-8, 9-12, 13-16, and 17-20 were determined, and the beams were appropriately designed for the respective floor grouping. The resulting coupling beam design demands, \( V_u \) (237kN (62kip) \( \leq V_u \leq 3,590kN \) (807kip)), design capacities, \( \phi V_u \) (295kN (66kip) \( \leq \phi V_u \leq 3,599kN \) (809kip)), and design procedures are reported in Fortney (2005).

7. DESIGN OF WALL PIER LONGITUDINAL STEEL
The flexural reinforcing steel for the wall piers was proportioned based on design axial and flexural demands. Design axial load-moment interaction curves were generated to verify the flexural and axial load-carrying capacities; biaxial loading was considered. To facilitate design, a fiber cross-sectional analysis software package called XTRACT (Imbsen and Associates, Inc.) was utilized. Consistent with the objective of this work aimed at establishing the potential of providing sufficient capacity of the wall piers with no special boundary elements, an unconfined concrete material model was assumed for all regions. Two curtains (an exterior curtain at each face) of uniformly distributed longitudinal steel were used for design as not to create any localized areas of concentrated normal forces in the wall section. The reinforcement ratios in the base wall piers range from 1.0% to 5.95%. A summary of the wall pier demands (including wall overstrength factors) and the resulting bars sizes, spacing, and reinforcement ratios is reported in Fortney (2005).

8. DETERMINING THE NEED FOR SPECIAL BOUNDARY ELEMENTS
8.1. Stress Calculations Based on ACI 318-08 §21.9.6.3
Provision ACI 318 (2008) §21.9.6.3 states that special boundary elements are required if extreme fiber concrete compressive stress, calculated using an assumed linear stress distribution over the depth of the section and gross section properties, exceeds 20% of the specified concrete compressive strength. For the LC1 and LC2 load cases (see Figure 3), the extreme fiber compressive stresses are calculated as follows:

Tension Wall:

\[
\sigma_{cc} = \frac{P}{A} + \frac{\sum V_b}{A} \times \frac{M_{xx}}{I_{xx}} + \frac{M_{yy}}{I_{yy}}
\]  
(2)

Compression Wall:

\[
\sigma_{cc} = \frac{P}{A} - \frac{\sum V_b}{A} \times \frac{M_{xx}}{I_{xx}} + \frac{M_{yy}}{I_{yy}}
\]  
(3)

Intermediate Wall:

\[
\sigma_{cc} = \frac{P}{A} + \frac{\sum V_b}{A} \times \frac{M_{xx}}{I_{xx}} + \frac{M_{yy}}{I_{yy}}
\]  
(4)

In Eqs 2, 3, and 4, \( \sigma_{cc} \) is the extreme fiber concrete compressive stress, \( P \) is the design axial load, \( \sum V_b \) is the summation of the beam shear capacities above the floor being considered, \( A \) is the gross cross-sectional area of the section, \( M \) is the design moment, and \( I \) is the moment of inertia of the gross wall section. Note that for the intermediate I-shape, \( \sum V_b = 0 \) as a result of geometric symmetry. For illustration, the calculated stresses over the height of Structure_01 at points A and C for LC1 (refer to Figure 3) are shown in Figure 4. The specified concrete compressive strength is 35MPa (5,000psi); hence, any compressive stresses exceeding 20% of this value, 7MPa (1,000psi) in this case, will result in mandatory special boundary elements. Additionally, §21.9.6.3 requires that where boundary elements are required, they may only be discontinued when the compressive stress falls below 15% of the specified compressive strength, or 5.3MPa (769psi) in this case. Therefore, as can be seen in Figure 4a, for Structure_01 per ACI 318 (2008) §21.9.6.3, special boundary elements would be required in the tension wall (point A) up to and including level 3. Referring to Figure 4b, special boundary elements would be required in the compression wall (point B) up to and including level 12. The calculated stresses in all of the eight structures considered in this study, using the provision in ACI 318 (2008) §21.9.6.3, are reported in Fortney (2005).
8.2. Compression Block Depth Determinations

(ACI 318-08 §21.9.6.2)

ACI 318 (2008) §21.9.6.2 requires special boundary elements where the depth of the compression zone exceeds the critical value given in Eqn 1. In order to investigate the application of this provision, the depth of the compression zone is required to be determined. Using the cross-section analyses generated with XTRACT, the depth of the compression zones were determined for the base wall piers in each of the eight structures for both load cases (LC1 & LC2). The ratio of $\delta_u/h_w$ in Eqn 1 can reasonably be taken as 0.007 for practical CCW configurations. For example, the maximum elastic interstory drift in the coupled direction in Structure_01 is $15\text{mm}$ ($0.591\text{in}$) ($C_d = 5$) and $h_w$ is equal to $2,740\text{mm}$ ($108\text{in}$) resulting in $\delta_u/h_w = 15\text{mm}/2,740\text{mm} = 0.0055 < 0.007$. The ratio, $\delta_u/h_w$, is less than 0.007 in all eight structures (see Fortney 2005). Therefore, $\delta_u/h_w$ is taken as 0.007 for all of the buildings considered in this study. Rearranging Eqn 1, special boundary elements will be required when the ratio of $c/l_w$ exceeds 0.24. Table 2 summarizes the compression zone depths in each of the base wall piers for each of the eight buildings considered in this study.

In most cases, $c/l_w$ exceeds 0.24. The depth of the compression block in the compression walls tends to be relatively large (in most cases the entire section is in compression) due to the large axial compression forces. As seen in Table 2, special boundary elements would be required in all of the eight buildings considered in this parametric study.

8.3. Compression Stress and Strain Determination from Cross-Section Fiber Analysis

As discussed previously, the objective of this study is to investigate the applicability of current ACI 318 (2008) provisions for assessing the need for special boundary elements in wall piers in CCW systems, and the possibility of reducing wall pier steel congestion common in special boundary elements. Accordingly, a fiber analysis program called XTRACT was used to compute concrete compression stresses and strains at locations A, B, C, D, E, and F shown in Figure 3. The

![Figure 4](image-url)  
**Figure 4.** Extreme fiber concrete *compressive* stresses in the wall piers of Structure_01 at (a) points A (tension wall) and (b) point C (compression wall), for LC1 ($1\text{MPa} = 145\text{psi}$). *Compression is negative

<table>
<thead>
<tr>
<th>Structure</th>
<th>Tension wall</th>
<th>Compression wall</th>
<th>Tension wall</th>
<th>Compression wall</th>
<th>I-shaped wall</th>
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<td>LC2</td>
<td>LC1</td>
<td>LC2</td>
<td>LC1</td>
</tr>
<tr>
<td>_01</td>
<td>0.25</td>
<td>0.32</td>
<td>1.00</td>
<td>0.73</td>
<td>0.41</td>
</tr>
<tr>
<td>_04</td>
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<td>0.32</td>
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<td>0.77</td>
<td>0.44</td>
</tr>
<tr>
<td>_07</td>
<td>0.21</td>
<td>0.66</td>
<td>1.00</td>
<td>0.75</td>
<td>0.27</td>
</tr>
<tr>
<td>_08</td>
<td>0.21</td>
<td>0.70</td>
<td>1.00</td>
<td>0.75</td>
<td>0.37</td>
</tr>
</tbody>
</table>

*Special boundary elements required for $c/l_w > 0.24$ – See discussion in the “Compression Block Depth Determinations” section of this paper

**Table 2.** $c/l_w$ ratios (results of fiber analyses)

![Table 2](image-url)
unconfined concrete material was considered to have zero tensile strength. Parabolic stress-strain relationships for an unconfined concrete model and an elasto-plastic model for steel reinforcement were considered.

The stability of the longitudinal reinforcing steel in the wall piers is assumed to be provided by the surrounding unconfined concrete. Concrete compressive failure for unconfined concrete was assumed to occur at a compressive strain equal to 0.004. The extreme fiber concrete compressive stresses, resulting from demands from LC1 and LC2, were determined using cross-sectional fiber analysis and then compared to the concrete compressive stresses computed per ACI 318(2008) §21.9.6.3, i.e., using gross section properties and an assumed linear distribution of stresses over the depth of the section.

In order to compare the concrete compressive stresses obtained from the two different methods (ACI §21.9.6.3 versus fiber analysis), the ratio of the compressive stresses determined from the fiber analyses to the compressive stresses determined from the ACI method, \( \sigma_{\text{Fiber}} / \sigma_{\text{ACI}} \), was computed. The resulting ratios for Points A and C for load case LC1 are shown in Figures 5 and 6. When referring to these figures, consider that when the ratio is equal to 0.0, the fiber analysis predicts a tensile concrete strain. Since the unconfined concrete model is assumed to have no tensile capacity, the ratio is 0.0. This condition reveals nothing concerning the evaluation of the ACI provision.

9. DISCUSSION

9.1. Design of CCW System Wall Piers and Effect of Large Axial Forces

The design of wall piers in coupled core wall (CCW) systems is driven by the loads on the tension wall. To ensure sufficient capacity of the wall piers (tension wall demands govern design of wall piers), a sufficient area of steel must be provided to resist the large axial tension loads on the tension wall. When sufficient reinforcement is provided to resist the tension wall loads, a significant amount of overstrength is provided for the loads on the compression wall. The ramifications of this type of “tension-driven” design are discussed with reference to Figure 7. This figure shows the demands on the compression wall and the tension wall plotted against two P-M diagrams for the wall piers at levels 1-4 in Structure_01. The larger P-M diagram corresponds to a maximum useable concrete compression strain (\( \varepsilon_c \)) of 0.003 (the value recommended by ACI), and the smaller one is for cases with a linear concrete stress distribution.

Figure 8 shows the stress-strain relationship for an unconfined 35MPa (5,000 psi) concrete. From this figure, the strain corresponding to 0.4\( f'c \), which is generally accepted as the upper limit for linear behavior of concrete, is 0.0005. Accordingly, the smaller P-M diagram in Figure 7 was generated for concrete strain equal to 0.0005. Note that at concrete strain of 0.003, the value for which the larger P-M diagram in Figure 7 was generated, the distribution of concrete stress is nonlinear (see Figure 8).
From Figure 7, it is evident that not only are the compression wall (CW) demands well within the “ultimate strength” P-M diagram ($\varepsilon_c = 0.003$) but also are just at or inside the “service load” P-M diagram ($\varepsilon_c = 0.0005$). Therefore, the compression walls have significant amount of reserve strength, and perhaps more importantly their behavior can adequately be captured by assuming a linear concrete stress distribution, which is consistent with current ACI provisions (§21.9.6.3). As a result, the concrete compressive stresses in the compression wall using the method permitted by ACI §21.9.6.3 correlate well with those from the fiber analysis method (refer back to Figures 4, 5, and 6). The base tension wall (TW) demand, on the other hand, falls outside of the service load P-M diagram (the point that lies outside of the service load P-M diagram).
load P-M, and just within the ultimate P-M is the base wall demand, the demand for which the wall pier was designed), and are actually just within the ultimate strength interaction diagram. Hence, a nonlinear concrete stress distribution is more reasonable for tension walls yet current code provisions (ACI §21.9.6.3) do not account for this behavior. The poor correlation of the concrete compressive stresses in the tension wall as computed by ACI §21.9.6.3 and fiber analysis is attributed to the nonlinear concrete response. Similar trends were consistently observed for all floors of the eight structures considered in this study.

Considering that the design of coupled core walls is driven by the demands on the tension walls, it is not appropriate to establish the need for special boundary elements in the wall piers of CCW systems by assuming linear concrete stress distribution. Other methods such as the use of fiber analysis are more appropriate.

9.2. Stress Comparisons - ACI 318-08 §21.9.6.3 versus Fiber Analysis

There is good correlation between the stresses calculated based on gross section properties and those from the more rigorous fiber analysis approach for all structures when considering the cantilever wall direction in LC2 (see Fortney 2005). However, when considering the tension wall stresses in the coupled direction, LC1 (point A for LC1 – see Figure 3), the ACI provision significantly underestimates the stress relative to that predicted using fiber analyses. The results shown in Figure 5 indicate that the ratio, $\sigma_{\text{Fiber}} / \sigma_{\text{ACI}}$, is as large as 4.25 (occurring in Structure_07). This trend illustrates that the ACI method (§21.9.6.3) significantly underestimates extreme fiber compressive stresses in the tension walls of coupled core wall (CCW) systems relative to fiber analysis, and can potentially lead to an unsafe design; i.e., special boundary elements may not be provided where needed if gross section properties are assumed. A similar observation can also be made from the graph shown in Figure 4(a) (for Structure_01) where it is shown that the compressive stresses predicted at Point A in the tension wall determined from the fiber analysis are significantly greater relative to the concrete compressive stresses predicted using the ACI provision.

Conversely, the two different methods have reasonable correlation at Point C (see Figure 3) as can be seen in Figures 4(b) and 6. Figure 6 shows that the ratio, $\sigma_{\text{Fiber}} / \sigma_{\text{ACI}}$, tends to be near 1.0. This is reasonable considering that Point C is located on a compression wall where the large axial compressive forces, combined with flexure, tend to develop linear stress distributions over the depth of the cross-section (as was discussed previously).

9.3. ACI 318-05 §21.9.6.2

This provision is based on a displacement-based design approach that considers continuous cantilevered walls with one flexure-critical section where axial loads due to frame action are not present and, as such, tensile normal stresses due to flexure may not be overcome by the compressive stresses resulting from gravity loads alone. The intent of this provision is to ensure that special boundary elements are provided when concrete strains, at factored loads, exceed a critical strain. For strain levels below a critical strain, significant concrete strength degradation and spalling of the concrete cover are not anticipated (Wallace 1994).
In this reported study, the critical compressive strain capacity was taken as 0.004. Using the XTRACT cross-sectional analysis software, the extreme fiber compressive strains at points A and C (see Figure 3), under factored loads, were determined at each floor of all of the eight structures. Plots of the resulting concrete compressive strains are provided in Figures 9-12. As can be seen in Figure 10, the concrete strain at point A, at level 1 in Structure_05, is 0.0018 which is the largest compressive strain for all of the structures. This strain is relatively moderate compared to the critical strain of 0.004. Referring to Figures 9-12, which plot the concrete compressive strains at Points A and C (see Figure 3), it is observed that, based on concrete compressive strain capacity (0.004), special boundary elements would not be required in any of the wall piers of the eight structures considered in this study.

**Figure 9.** Extreme fiber concrete *compressive strains determined from fiber analyses for Point A for LC1 (one-cell structures)

*Compression is negative

**Figure 10.** Extreme fiber concrete *compressive strains determined from fiber analyses at Point A for LC1 (two-cell structures)

*Compression is negative
10. CONCLUSIONS

- The ACI 318 (2008) §21.9.6.2 provision for determining the need for special boundary elements, based on the location of the neutral axis, is not appropriate for wall piers in coupled core wall (CCW) systems. Even in the case of compression walls where no net tension is developed, and the wall pier section may be entirely in compression, it has been shown that extreme fiber compressive strains are moderate and well below an acceptable unconfined concrete compressive strain equal to 0.004.

- The ACI 318 (2008) §21.9.6.3 provision for determining the need for special boundary elements, based on the state of stress at the extreme compression fiber, is not appropriate for wall piers in CCW systems. The axial forces imparted to the wall piers by the coupling beams are significant and the tension wall stresses can be profoundly underestimated in cases where large net axial tension forces are present and the effects of cracked sections and nonlinear material behavior are significant.

- When determining the need for special boundary elements in wall piers in CCW systems, a more
refined analysis, such as fiber analysis, is required due to nonlinear stress distributions at ultimate loads. The state of extreme fiber concrete compressive strain at design loads should be considered. Furthermore, considering that wall piers in CCW systems cannot be assumed to have a single flexure-critical section over the height of the building, evaluation of the need for special boundary elements should be made at each range of floor levels where contra flexure or change in wall pier reinforcement exists. Where compressive strains are expected to be below the unconfined concrete compressive strain capacity, special boundary elements are not required.

- The results of this study are predicated on the assumptions made regarding parameters and material models. For example, changes in the assumptions of cracked sections, elastic analysis, and parameters used to define the constituent material models could lead to different results. However, it should be noted that the assumptions made during the course of this reported study are considered to be reasonably accurate to the assumptions made in structural design offices.

REFERENCES

ACI Committee 318. (2005). *ACI 318-05/ACI 318R-05: Building Code Requirements for Structural Concrete and Commentary*, American Concrete Institute, Farmington Hills MI.


Fortney, P.J. (2005). *The Next Generation of Coupling Beams*, PhD Dissertation, Department of Civil and Environmental Engineering at the University of Cincinnati, Chapter 5 and Appendix C (Chapter and Appendix made available by contacting Fortney at pat.fortney@udayton.edu)


