

Practical Lessons for Concrete Wall Design, Based on Studies of the 2010 Chile Earthquake



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SUMMARY:

The Maule Chile Earthquake of 27 February 2010 caused damage to several mid-rise and high-rise concrete wall buildings, and the performance of these buildings offers practical lessons for structural engineering design. Post-earthquake reconnaissance teams observed damage including buckling of reinforcing bars and concrete crushing at wall boundaries, overall wall buckling, and damage resulting from building configuration issues such as discontinuities and coupling from slabs, beams, spandrels, and stairs. This damage raises important questions about the physical mechanisms that lead to bar buckling, concrete crushing, and wall buckling. How engineers interpret the damage can lead to different conclusions about the detailing necessary to provide ductility in flexure-governed walls. A team of practitioners and researchers is developing recommendations related to the behaviors described above through the ATC-94 project. This paper summarizes current work and preliminary findings, including post-earthquake observations, analytical studies of whole buildings and individual structural elements, and concepts for potential changes to improve design practice.

Keywords: Chile, concrete walls, buckling, crushing, configuration

1. INTRODUCTION

The magnitude 8.8 earthquake that occurred on 27 February 2010 off the coast of the Maule region in central Chile is one of the seven largest earthquakes in recorded history. Peak ground accelerations as great as 0.9g were recorded in some locations (and greater than 0.3g in many locations), exceeding the elastic design response spectra from the Chilean building code in several cases (Boroschek 2010, EERI 2010, NCh433 1996). Also, soil amplification effects at certain sites caused peaks in the measured acceleration spectra at longer periods, particularly in the city of Concepcion and also in Viña del Mar (Boroschek 2012). There was a long duration of strong shaking, lasting approximately two minutes in the Concepcion area.

As a result of frequent historic seismic activity in the region, building codes in Chile include consideration of seismic effects, and building practice includes seismic-resistant construction. Because modern Chilean practice has been largely modeled after U.S. practice (with some important differences), investigations into the performance of engineered structures during the 2010 earthquake are important to future seismic design and construction practice in both the United States and Chile.

In response to the earthquake, several U.S. organizations sent reconnaissance teams to Chile to gather information that could be used to study implications for U.S. practice. Most mid-rise and high-rise buildings in the affected areas were constructed with seismic-force-resisting systems consisting of reinforced concrete structural walls. Similar construction is also prevalent in regions of high seismicity in the Western United States. Reconnaissance teams observed many instances of structural damage to reinforced concrete walls that could be relevant to U.S. building codes and standards.

Recognizing the potential value to U.S. design practice, the National Institute of Standards and

Technology (NIST) initiated a series of projects intended to study the effects of the earthquake and incorporate lessons learned into U.S. engineering practice. The objectives of the ATC-94 project are to: (1) evaluate critical issues in the design of reinforced concrete walls; and (2) develop improved wall design requirements. Work includes studying wall failure modes and observed damage and conducting focused analytical studies on the potential causes of observed behavior. Major topics of investigation include confinement triggers, plastic hinge length, bar buckling, overall wall buckling, effects of coupling, effects of irregularities, interaction with floor structures and other elements of the gravity system, and assessment of advanced analytical simulations of walls.

Figures 1.1 through 1.3 show examples of observed earthquake damage exhibiting some of the key behaviors studied in this project. The types of earthquake damage, studies being performed, and potential lessons are described in subsequent sections of this paper.

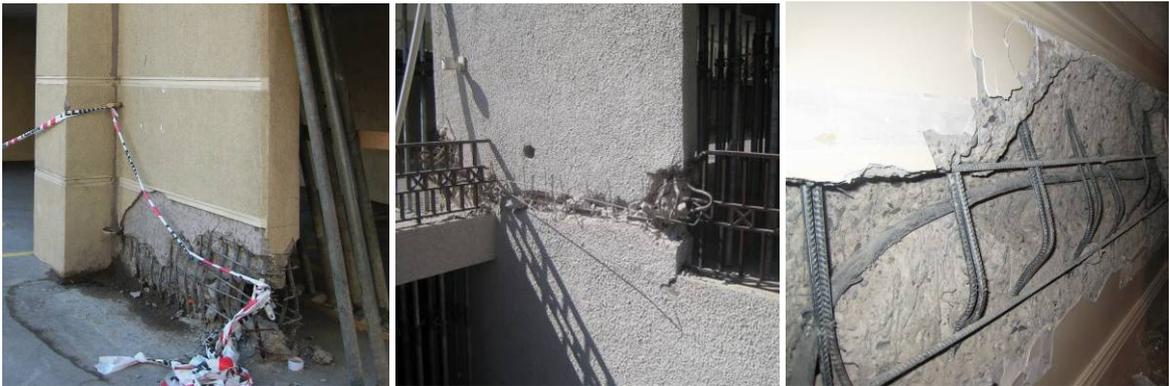


Figure 1.1. Damage to wall boundaries: longitudinal bar buckling and concrete crushing.



Figure 1.2. Damage concentrated at discontinuities (photo (c) by Professor Patricio Bonelli)



Figure 1.3. Damage to coupling elements (a) slabs (b) beams (c) stairs.

2. WALL BOUNDARY DETAILING

Much of the damage observed in concrete walls of mid-rise and high-rise buildings in the Chile earthquake consisted of buckling of longitudinal (vertical) reinforcing bars and crushing of concrete near the base of the walls. This type of failure was most severe at wall boundaries, but damage also tended to propagate over much of the length of the wall as shown in Figure 1.1 and Figure 1.2c. It is an undesirable failure mode because it can lead to strength degradation and irreparable damage. Such damage may have contributed to the total collapse of one building in Chile.

This type of damage results from flexural compression and/or cyclic tension and compression at the extreme fibers a wall section. The details of the physical mechanisms leading to bar buckling and concrete crushing are complex, and two somewhat different interpretations tend to be offered to explain the damage to flexure-governed walls in Chile, as summarized in Table 2.1. The first explanation proposes that failure is initiated by concrete spalling, primarily because of compression strain demands. The second proposes that buckling of longitudinal reinforcing bars initiates failure, because of tensile strain followed by a reversal into compression.

Table 2.1. Two possible explanations for wall boundary failures in the 2010 Chile Earthquake

Observation	<i>Spalling-first interpretation</i>	<i>Buckling-first interpretation</i>
Cause/initiation of failure	The failures are associated with flexural <i>compression</i> . Damage occurs because compression strain demand exceeds compression strain capacity of the concrete. Strain capacity may be smaller than traditional assumptions. Bar buckling occurs after spalling and crushing, as a consequence of the flexural compression failure. High axial load contributes to the failures.	The failures are typically associated with flexural <i>tension followed by compression</i> . Tensile strain stretches bars prior to a reversal into compression that causes buckling. Buckling in monotonic compression is also possible, but prior tensile strain makes bars susceptible to buckling at smaller compression strain. Bar buckling occurs prior to any significant spalling or crushing of cover concrete, with the bar buckling itself helping to spall off the cover concrete and also reduce the effective confinement of the concrete core. Neutral axis depth (in both tension and compression) is a more important variable than axial load.
Vertical concentration of damage	Strain demand is large because crushing occurs over a very short height of wall.	Once bar buckling and spalling occur, compression strain concentrates in the concrete at the reduced section caused by the buckling, which is then heavily damaged by cycles of compression.
Horizontal propagation of damage	With continued cycling, damage progresses further into the wall section.	A large tension neutral axis depth makes a large depth of the wall vulnerable to bar buckling. With cycling, damage progresses further into the wall section.
Implication for design requirements	The cause of wall damage in Chile is a lack of adequate transverse reinforcement to provide confinement. It may not even be possible to provide enough confinement in thin sections because the core area is small and the pattern of spalling indicates that compression strain concentrates over a short height. Thus moderate amounts of well-detailed confinement may not improve performance.	The cause of wall damage in Chile is a lack of adequate transverse reinforcement to restrain bar buckling. Moderate amounts of well-detailed confinement, (e.g., spaced at $6 d_b$) should restrain longitudinal bars from buckling. If bars are restrained from buckling, compression strain can distribute over a greater height, and the strain demands will not be so high. Thus performance would be improved.

The two interpretations lead to different conclusions about the cause of this damage and its implications for code provisions. Both interpretations assume that the damage initiates in the extreme boundaries of the wall section, where strain (either in tension or compression) is highest. In either scenario, the propagation of damage into the wall depth could be the result of subsequent cycles after the boundaries have lost the capacity to transfer compression force. Because the tension zone of a wall is generally deeper than the compression zone, it may be more likely to see this damage throughout a

wall section in the buckling-first scenario.

Dhakai and Maekawa (2002a, 2002b) suggest that cover concrete spalling can be influenced both by compression strain and bar buckling, with compression strain initiating cracking, and bar buckling causing cracks to widen and eventually separate cover concrete from the core concrete. They propose that the buckling behavior of reinforcement depends on the expression $L/D\sqrt{f_y}$, where L is the buckling length, D is the diameter of the longitudinal bar, and f_y is the yield strength of the longitudinal bar. The buckling length L is related to the spacing of transverse ties, but it may be greater if ties are not stiff enough (too small or not enough ties per longitudinal bar) to provide adequate restraint against buckling. This type of reinforcement buckling (as opposed to elastic Euler buckling) occurs after bars have begun to yield in compression, and it is affected by strain history because bars may have undergone plastic tension strain prior to yielding in compression. Rodriguez et al (1999) also discusses the relationship of tension strain, tie spacing, and buckling behavior of longitudinal reinforcing bars under cyclic loading.

Some evidence from the Chile earthquake indicates that bar buckling was a key cause of damage. For example, all of the serious flexural damage to walls reported has included buckled bars. The authors are not aware of reports of spalling without buckled vertical bars. Also, all of the damaged walls reported had inadequate transverse reinforcement, similar to Figure 2.1a. Some engineers in Chile state that they do use well-detailed transverse reinforcement and that such walls did not suffer any damage. In the spalling-first scenario one would expect some damage even to well-detailed walls.

The buckling-first interpretation is also consistent with behavior observed in the test specimens by Thomsen and Wallace (2004), shown in Figure 2.2. T-shaped wall specimens with different amounts of wall boundary ties were subjected to cyclic static loading with increasing displacement. The specimen in Figure 2.2a, with widely-spaced hoops and cross-ties extending only a short distance into the wall web, showed bar buckling occurring suddenly at 1.25% drift, without significant prior spalling. The specimen in Figure 2.2b, with more closely-spaced hoops and cross-ties extending further into the wall web, showed more ductile response, achieving 2.5% drift before the wall web began to buckle out-of-plane.

The ATC-94 project includes analyses of damaged and undamaged walls from several different Chilean buildings in an attempt to determine which scenario most accurately describes the observed damage in Chile and in experimental tests. Both interpretations lead to a conclusion that inadequate ties in the boundary zones make a wall more likely to experience damage of this type. The implications of the spalling-first scenario would lead to requirements for a greater area of confinement ties in the compression boundaries. The implications of the buckling-first scenario would lead to close spacing of ties (not necessarily greater tie area) and would possibly imply ties further into the section; the emphasis of design for flexural walls such as those in Chile would focus on restraining bars from buckling, because if this is done the questions of strain demand and strain capacity may be less critical.

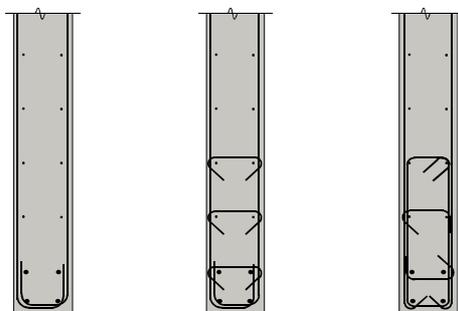


Figure 2.1. Wall boundary detailing (plan sections): (a) No hoops or cross-ties; typical of walls with observed damaged in the 2010 Chile Earthquake. (b) Cross-ties with 135-degree hooks; authors are not aware of any damage observed to walls with this type of detailing in the Chile Earthquake. (c) Hoops and cross-ties with 90-degree and 135-degree hooks; satisfying current ACI 318 detailing requirements.

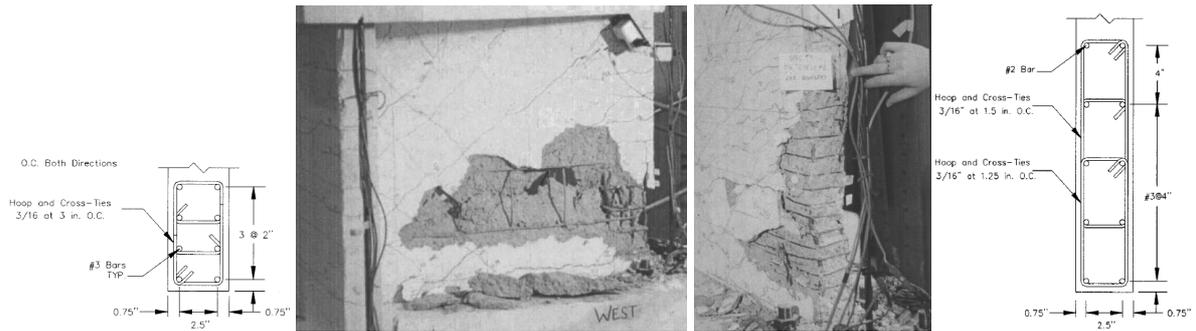


Figure 2.2. Test specimens from Thomsen and Wallace (2004): (a) Hoops and cross-ties spaced at $8d_b$, extending $0.15L_w$ into wall web. (b) Hoops and cross-ties spaced at $4d_b$, extending $0.35L_w$ into wall web.

3. OVERALL WALL BUCKLING

Overall wall buckling refers to the buckling of a portion of a wall section out-of-plane (as opposed to buckling of individual reinforcing bars), as a result of in-plane wall flexure during an earthquake. The buckling is typically limited to an end region of the wall where vertical tension and compression strains from in-plane wall flexure are greatest. (Gravity loads can contribute to compression demands, but earthquake loads are the primary cause of the type of buckling described here.)

Prior to the 2010 Chile Earthquake, this phenomenon had been observed in experimental tests (Figure 3.1c,d), but the authors are not aware of any reports of this behavior in an actual earthquake. Relative to other types of observed wall damage in the Chile Earthquake, the number of instances of overall wall buckling is relatively few (Figure 3.1a), and some engineers question whether or not this behavior is distinguishable from concrete crushing in certain instances. These instances were often associated with significant residual drift in the building. This behavior was also observed in the 2011 Christchurch, New Zealand, Earthquake (Figure 3.1b).

Current U.S. building codes do not include a requirement for minimum wall thickness to prevent overall wall buckling. Previously, the Uniform Building Code (ICBO 1997) required a minimum thickness of $1/16$ times the wall unbraced height. FEMA 306 (ATC 1999), which references Paulay and Priestley (1992, 1993) and SANZ (1995), presents similar recommendations and provides more detail relating the assumed unbraced height to story height as well as plastic hinge length. FEMA 306 classifies overall wall buckling as a behavior mode of moderate ductility capacity, given that it is preceded by some cycles of ductile flexural behavior before buckling and consequent strength degradation.

A notable feature of the examples shown in Figure 3.1 is that the buckling length of the wall is less than the clear story height. Also, the wall in Figure 3.1b is relatively thick, apparently satisfying the criteria listed above from the Uniform Building Code. These observations indicate that other factors, besides the thickness and clear height of the wall, can be influential. Tension strain has been suggested as an important factor because this causes concrete cracking and, upon load reversal, requires vertical bars to carry compression loads. One curtain of reinforcement will inevitably yield before the other, which could lead to wall buckling if cracks are large enough relative to the thickness of the wall.

To improve performance, potential modifications to design practice could include providing a minimum wall thickness at the compression boundary of a wall, as a function of the unsupported wall height and/or tensile strain demand in the region of potential plastic hinging. The thickness requirement could also depend on other variables such as unbraced wall length, axial load, or neutral axis depth. The ATC-94 project includes studies of experimental tests and the damaged buildings shown in Figure 3.1 to verify the causes and identify design limitations to avoid this behavior.



Figure 3.1. Examples of overall wall buckling: (a) 2010 Chile Earthquake (photo by Professor Jack Moehle) (b) 2011 Christchurch Earthquake (Kam et al 2011) (c, d) Tests by Goodsir (1985)

4. BUILDING CONFIGURATION

In addition to detailing issues for individual structural components (discussed in previous sections of this paper), damage from the Chile Earthquake also highlights several building configuration issues that can affect component demands and overall structural performance. A range of different issues led to damage in the earthquake, including discontinuities such as those shown in Figure 1.2 and others. The most frequently observed issues generally fall into the following categories: wall coupling, wall length discontinuities, wall cross-section, and other irregularities such as changes in wall location or shape from story to story. These are each described briefly in the following subsections.

4.1. Wall coupling

Damage was observed from coupling due to slabs, beams, spandrels, and stairs transmitting forces between distinct structural walls (Figure 1.3). Where coupling is the intended mechanism of response and walls and coupling beams are detailed accordingly, coupling beams can be an effective means for dissipating energy. However, if coupling effects are not explicitly accounted for in design, unintended behavior may occur in the coupling elements, or the coupling can increase the shear demand and neutral axis depth of coupled walls beyond that provided for in the design. This can potentially lead to the wall boundary damage discussed previously.

Potential improvements to design practice include explicitly accounting for the coupling elements in the seismic analysis and design, or detailing them to minimize interaction with the designated seismic force-resisting elements.

4.2. Wall length discontinuities

Abrupt changes in wall length led to damage in cases where shorter walls occurred above longer walls (Figure 1.2b), and also where longer walls occurred above shorter walls (Figure 1.2a and 1.2c). In the example of Figure 1.2b, decreased wall length also coincides with a decrease in wall thickness and reinforcement. This discontinuity and the presence of weak shear-critical wall piers contributed to a partial story collapse at that level. In examples such as Figure 1.2c, wall length decreases at lower levels to accommodate drive-aisles and parking constraints, while apartments above extend toward the property line. Figure 4.1 describes observed damage and associated design characteristics in this type of building configuration.

The nonlinear finite element model shown in Figure 4.2 was developed by working groups on the ATC-94 project to analytically capture behavior of an example building that includes the configuration issues described in Figure 4.1 and that suffered damage in the Chile Earthquake. When subjected to

recorded earthquake motions, the model predicts concrete crushing failure, initiated in the extreme fiber of the wall at ground level (Figure 4.2c). The strain distribution along the base of the wall is not linear; compression strains are much greater near the end of the wall. With successive cycles, the failure spreads along the length of the wall, with little vertical distribution of strain. After several cycles, vertical-load-carrying capacity is lost along much of the wall length and the building collapses. More details of the analysis are provided by Telleen et al (2012).

Possible recommendations for improving design practice could include avoiding vertical discontinuities near the intended plastic hinge region of walls, adjusting analysis assumptions or limits on concrete compression strain, using special detailing requirements to improve compression and cyclic behavior of wall boundaries, and providing reinforcement to rationally resolve forces at discontinuities.

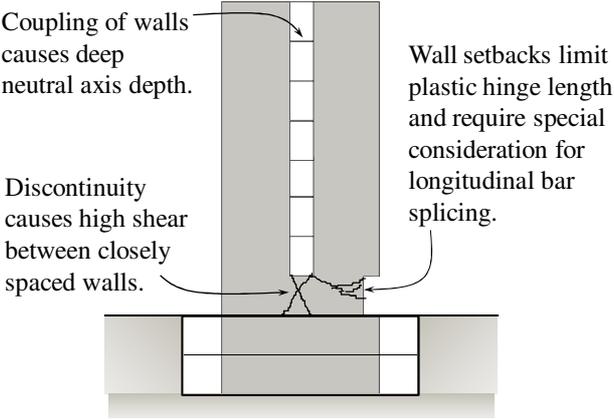


Figure 4.1. Wall configuration issues

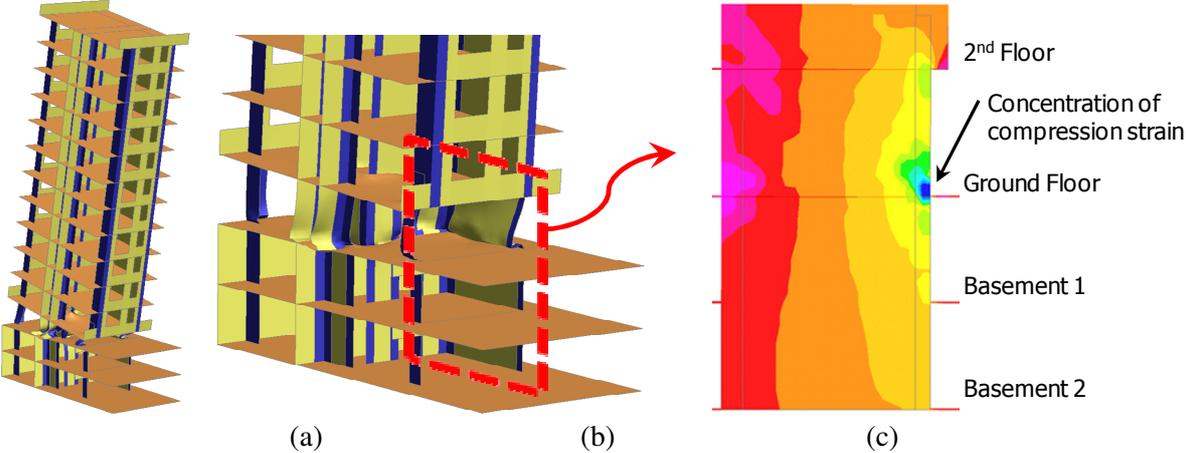


Figure 4.2. Analysis of wall with vertical discontinuity (a) model deformed shape at failure (b) close-up of lower stories (c) partial wall elevation showing vertical strain concentration at ground floor. (models and figures by Yuli Huang and Michael Willford)

4.3. Wall cross-section

Several instances of damage to wall boundaries occurred in walls with flanged cross-sections such as T-shapes and L-shapes (Figure 1.1a, Figure 3.1a). In such configurations, the neutral axis depth is different depending on the direction of bending (Figure 4.3c). Paulay and Priestley (1992) discuss this effect in more detail, noting that a large neutral axis depth can lead to limited curvature ductility unless adequate confining reinforcement is provided in the critical compression regions of the wall. When the

flange is in compression, the neutral axis depth is shallow, and much of the web reinforcement is strained in tension. When the direction of shaking reverses so that the flange is in tension, the neutral axis depth is deep, imposing large compressive strains at the extreme fiber of the web and with compression extending a significant distance along the length of the wall. This type of behavior is consistent with damage such as that in Figure 1.1c, where vertical bars exhibit buckling along much of the length of the wall.

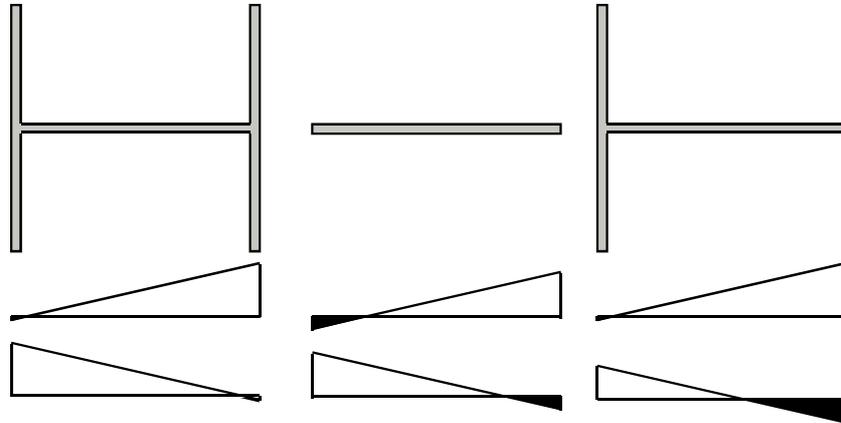


Figure 4.3. Neutral axis depth for wall sections (assumes $P = 0.1A_g f'_c$, $\rho = 0.0025$):
 (a) I-shaped $0.02L_w$ (b) rectangular $0.2L_w$ (c) T-shaped $0.4L_w$

5. CAPACITY DESIGN AND STRENGTH HEIRARCHY OF STRUCTURAL ELEMENTS

Reviewing the damage in Chile compared to current trends for more transparent “performance-based” code requirements, Bonelli et al (2012) have questioned whether performance-based design can be effectively applied to structures if the design process does not check explicitly for a suitable ductile mechanism of behavior. In some cases, such a check can be performed with relatively simple calculations.

The ATC-94 project includes studies of lateral-force-resisting elements, such as cantilevered and coupled walls to investigate how designers can best evaluate the expected behavior of these elements using simplified approaches as well as more sophisticated analytical models. The following example describes a study of a pier-spandrel system to illustrate the concepts of evaluating the expected mechanism of behavior for a structure.

The pier-spandrel system shown in Figure 5.1a is a portion of a high rise building (Figure 1.2b) that suffered severe damage and partial collapse in the Chile Earthquake. On the elevation shown (which did not collapse) the wall piers suffered severe shear damage when constrained between deep spandrels, while the spandrels suffered flexural damage at the ends where they connect to heavier piers of the end bays. Hand calculations, described in more detail in Telleen et al (2012), confirm that piers are shear-governed, and that the governing plastic mechanism for the system is as shown in Figure 5.1b. These calculations determine the expected behavior mode of each pier and spandrel by comparing their flexural and shear strength, and then considering several potential plastic mechanisms for the system and identifying the mechanism corresponding to the least lateral resistance.

Given that this behavior mode and mechanism are reasonably predictable by straightforward engineering calculations, the question can be considered whether design codes should require engineers to identify the governing behavior mode and mechanism, using capacity design principles. An additional issue is that US and Chilean codes currently make only a small distinction (via the strength reduction factor) in the assumed ductility capacity between shear versus flexural behavior. (For flexure, ACI 318 applies a strength reduction factor of 0.65 to 0.9, depending on whether the section is tension-controlled or compression-controlled. For shear, the strength reduction factor is 0.6

or 0.75, depending on whether the member is flexure-controlled or shear-controlled.) Shear failure of walls and wall piers is generally less-desirable than flexural yielding because shear failure tends to exhibit less ductility and can be associated with a concentration of lateral deformation and damage. For wall piers with slender aspect ratio, ACI 318-11 includes some new requirements similar to column design provisions. FEMA 306 (ATC 1999) and Maffei et al (2000) outline a more general process for identifying the governing mechanism of lateral deformation of various building configurations, similar to the example described above.

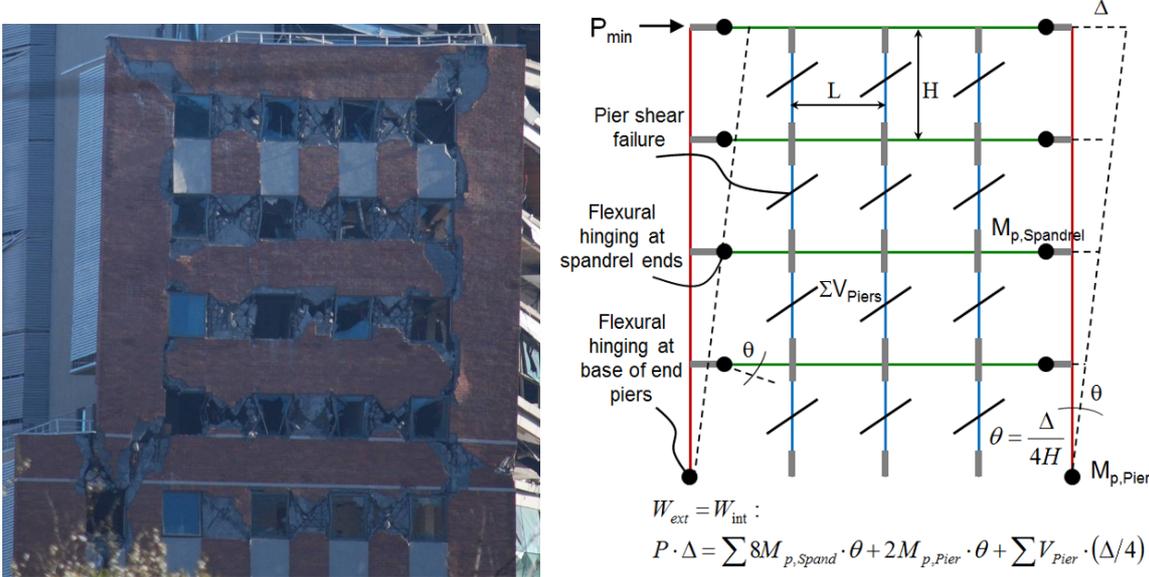


Figure 5.1. Pier-spandrel system: (a) observed damage (photo from EERI team) (b) plastic mechanism observed and validated by calculation (figure by Ady Aviram and Dominic Kelly).

6. CONCLUSIONS

The 2010 Chile earthquake offers a valuable opportunity to learn from damage caused to modern engineered structures and to improve the practice of structural engineering design. In particular, the earthquake demonstrated that flexure-governed walls can exhibit limited ductility when their behavior is affected by buckling of reinforcing bars, crushing of concrete, and overall wall buckling. Damage to wall boundary elements, especially in concrete walls without transverse hoops or crossties, raises important questions about the physical mechanisms that lead to bar buckling and concrete crushing, and the detailing necessary to provide ductility in flexure-governed walls. In addition to detailing, building configuration also plays a key role. Concentrations of tensile and compressive strain demands in walls can result from T-shaped and L-shaped wall cross-sections, wall setbacks, wall coupling, and other discontinuities.

Chilean engineers have responded to the earthquake by implementing several changes to design codes including modifying design spectra, changing the triggers that determine when special boundary zone detailing in walls is required, limiting compressive strains in walls, imposing minimum thickness requirements for wall boundaries, and improving detailing provisions for wall boundaries and shear reinforcement. Certain code modifications were initially adopted in Chile and later modified based on continuing discussions and response from the design community. The ATC-94 project team is considering where US code changes are warranted, both to the ACI 318 requirements for concrete, and to the ASCE-7 requirements for the classification of concrete seismic force-resisting systems and the specification of earthquake force and displacement demands.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the NEHRP Consultants Joint Venture (a partnership of the Applied Technology Council and Consortium of Universities for Research in Earthquake Engineering), under Contract SB134107CQ0019, Earthquake Structural and Engineering Research, issued by the National Institute for Standards and Technology (NIST), for funding the ATC-94 project described in this paper. In particular, we appreciate Steve McCabe for providing valuable insight for pursuing project objectives. The authors also appreciate the close collaboration with the ATC-94 project team: Professor Patricio Bonelli, Dominic Kelly, Professor Dawn Lehman, Professor Laura Lowes, Professor Jack Moehle, Professor John Wallace, Michael Willford, Begoña Aguirre, Ady Aviram, Anna Birely, Chris Hilson, Yuli Huang, and Pablo Parra, as well as project reviewers Professor Helmut Krawinkler, S.K. Ghosh, Professor Mete Sozen, Derrick Roorda, and Professor Tara Hutchinson. We appreciate the cooperation and insights provided by Professor Leo Massone and Sergio Contreras. We are saddened by the passing of Professor Helmut Krawinkler, and we are grateful to have had the opportunity to learn from him as a teacher and collaborator, and for his contributions to the practice of earthquake engineering.

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