LESSONS FOR CONCRETE WALL DESIGN FROM THE 2010 MAULE CHILE EARTHQUAKE

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ABSTRACT: Damage to mid-rise and high-rise concrete wall buildings caused by the 2010 Chile Earthquake offers a rare and valuable opportunity to study buildings in detail to gain practical lessons for structural design. Observed damage includes concrete crushing and reinforcing bar buckling in wall boundary elements, overall wall buckling, and damage resulting from configuration issues such as discontinuities. Through the ATC-94 project, a team of researchers and practitioners is developing recommendations for modifying design practices based on studies of damaged Chilean wall buildings.

Key Words: 2010 Maule Chile Earthquake, concrete wall, boundary element, buckling, crushing, shear, configuration

INTRODUCTION

The Maule Chile earthquake of 27 February 2010 subjected many engineered structures to strong earthquake shaking, and it presents opportunities to learn from the seismic performance of these buildings. The earthquake was large (Mw 8.8) with a long duration of strong shaking (two minutes in some locations), and in many cases buildings performed well. However, severe damage from ground shaking occurred in some buildings, including several mid-rise and high-rise concrete wall structures housing apartments. Post-earthquake reconnaissance teams reported that most buildings of this type use thin concrete walls—typically 200mm thickness for buildings up to 16 stories and 250mm up to 25 stories—as the primary gravity and lateral-force-resisting elements, and that the dimensioning, detailing, and configuration of these walls may have contributed to the damage sustained in the earthquake (EERI 2010, Cowan et al 2011).

The availability of complete structural drawings for many of the damaged buildings, designed to modern building codes, provides a rare and valuable opportunity for study. Such information has typically not been available after earthquakes in the US and other countries. The structural drawings and damage documentation enable quantitative studies that can be used to advance knowledge in the field of structural engineering.

Similarities exist between the United States and Chile in terms of many building code provisions, seismic hazards, and urban environment, so collaborative research on these topics offers potential benefits for both countries. For example, the Chilean building code, in place during the construction of many of the earthquake-affected buildings, incorporates many of the concrete design provisions from the U.S. standard ACI 318. One notable exception is that the provisions for special boundary elements in ACI 318 were not included in the Chilean code (INN 1996) until recently.

Following the earthquake, representatives of several U.S. earthquake engineering organizations met with Chilean researchers and practitioners and produced a list of potential engineering study topics that could lead to recommendations for improved design provisions based on information from the earthquake (Moehle 2010).

To study some of these items, a team of practitioners and researchers is collaborating through the Applied Technology Council (ATC) ATC-94 project "Seismic Performance of Reinforced Concrete Wall Buildings in the 2010 Chile Earthquake." The project objective is to evaluate critical issues in the design of reinforced concrete walls and recommend revisions to design requirements where appropriate.

This paper summarizes preliminary findings from the project, including post-earthquake observations, structural seismic behaviors being studied, and concepts for potential changes to building codes and design practices.

PROJECT ORGANIZATION

The ATC-94 project team consists of practitioners and researchers organized into working groups to conduct problem-focused studies on specific topics (Fig. 1). Working group studies draw on observed damage (or lack of damage) in several different mid-rise and high-rise concrete wall buildings, available information about detailing and construction in those buildings, past data from related experimental testing and research, and analytical studies. Studies make use of tools commonly used in engineering design offices as well as more advanced analysis tools. Analytical tasks include studies of individual concrete elements, studies of multi-story walls, and studies using full-building analyses.

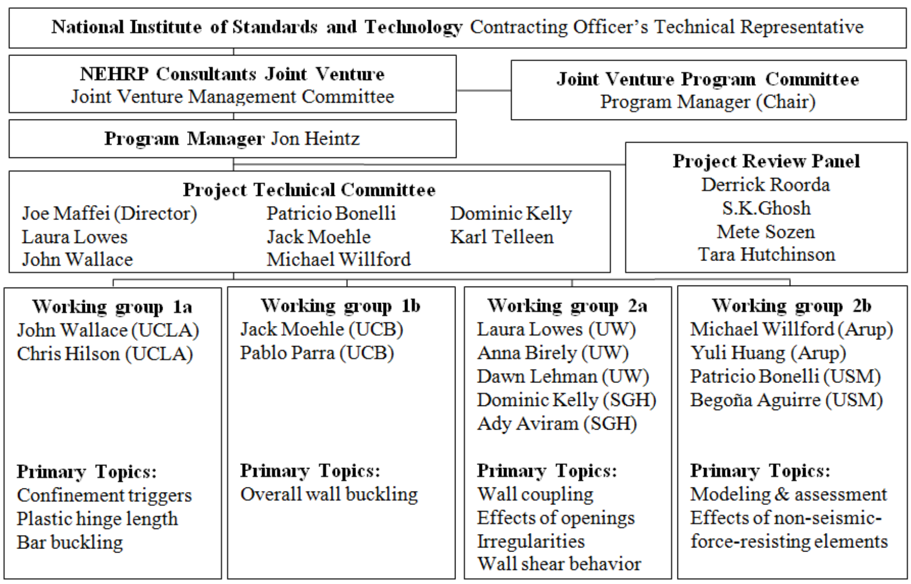


Fig. 1 ATC-94 project team

The final deliverable for the project will be a report, expected in late 2012, describing certain structural seismic behaviors observed in the Chile Earthquake, practical and theoretical understanding of the behaviors studied, and recommendations for modifying design practice and code provisions to improve the seismic performance of concrete wall buildings.

STRUCTURAL BEHAVIOR MODES STUDIED

Post-earthquake reconnaissance teams observed damage to buildings of a variety of construction types. In concrete buildings, which constitute most mid- and high-rise buildings in Chile, observed behavior modes included:

- damage to wall boundary areas including bar buckling, bar fracture, concrete crushing, and overall wall buckling
- damage concentrated at wall setbacks and flag-shaped walls
- bar splice failure
- wall damage extending into basement levels
- apparent plan torsion effects
- coupling of concrete walls from slabs, beams, and spandrels
- interaction of stairs with lateral-force-resisting systems
- shear failure of concrete walls and wall piers
- damage to walls with irregular openings
- soil-structure interaction

Certain of these behaviors are less desirable than others in terms of seismic safety, reliability, and repairability. Several of these behaviors have also been observed in experimental tests, which provide measured data to complement and compare to earthquake observations. In the ATC-94 project, discussions of study objectives led to questions including the following:

- Is it possible, as designers intend, for compression-governed walls to develop distributed yielding (over a certain plastic hinge length)? Should concrete walls be required to be tension-controlled in flexure (to preclude compression failure from flexure and axial loads)?
- What are the effects of earthquake duration on building performance? Does it depend on the behavior mode of the building?
- Should the seismic response modification factor (***R*** in U.S. building codes, ASCE 2010) depend on the expected behavior mode of a structural system rather than just the construction type?
- For pier-spandrel systems, should there be a code requirement to ensure strong-pier/weak-spandrel behavior, similar to current requirements for strong-column/weak-beam?
- How can engineers and society confront the challenges of demolishing tall buildings in an urban setting that have suffered severe earthquake damage?
- What causes a building to go from extreme damage to collapse? Are analysis methods capable of distinguishing between these limit states?
- What strategies have engineers and building owners used to decide what damage is repairable and what requires demolition?

The ATC-94 project focuses on the following selected behaviors, with the objective of taking meaningful steps to advance the practice of structural engineering:

Damage to wall boundary elements, including concrete crushing and/or buckling of longitudinal reinforcing bars (Figs. 2a, 3a)

These phenomena result from flexural compression and/or cyclic tension and compression. They can be undesirable failure modes because they can lead to strength degradation and irreparable damage. To improve performance, potential modifications to design practice could include providing transverse reinforcement ties at a close spacing and/or providing an increased area of transverse reinforcement in wall boundary elements. Wall sections could also be designed to be governed by tension yielding.

Different interpretations of this damage could lead to different design implications, as discussed in the next section.

Overall wall buckling (Figs. 2b, 3b)
This phenomenon consists of buckling of the wall section (as opposed to individual reinforcing bars) out-of-plane, resulting from flexural compression and/or cyclic tension and compression. Prior to the 2010 Chile earthquake this behavior mode had been observed in experimental tests but had not been reported in an actual earthquake. This behavior was also observed in the 2011 Christchurch, New Zealand, Earthquake. It can be an undesirable failure mode, particularly in regards to repairability of structures. 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 in the region of potential plastic hinging. The thickness requirement could also depend on other variables such as unbraced wall length, axial load, neutral axis depth, or expected strain demand.

Damage resulting from building configuration issues (Fig. 2c)
Coupling from slabs, beams, spandrels, stairs, and other outrigger-type elements can cause damage to these elements and can also increase shear demand in walls. Potential improvements to design practice include accounting for these elements in the seismic analysis and design, or detailing them to minimize interaction with the designated seismic force-resisting elements.

The following sections describe preliminary investigations related to some of these behavior modes. Full findings for the project will be described in the project report.

Fig. 2 Damage from Chile Earthquake (a) Damage to a wall boundary element (b) Overall wall buckling (photo by Prof. Jack Moehle) (c) Damage at wall discontinuity

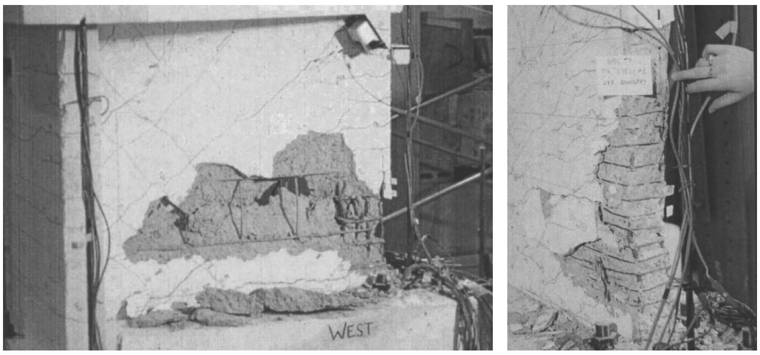


Fig. 3 Test specimens from Thomsen and Wallace (2004) (a) Wall boundary element with transverse hoops spaced at $8d_b$ exhibited longitudinal bar buckling and concrete crushing at 1.25% lateral drift. (b) Wall boundary element with hoops spaced at $4d_b$ exhibited more ductile behavior until initiating overall wall buckling at 2.5% drift.

WALL FLEXURAL FAILURES—TWO POSSIBLE INTERPRETATIONS

One of the key behaviors observed in the Chile earthquake is damage to multi-story walls near the base of the building, exhibiting buckled vertical reinforcement and crushed concrete concentrated over a relatively short height of wall. This type of failure was often not concentrated only at the boundaries, but was instead seen to propagate over much of the length of the wall as shown in Figs. 2a and 3a.

In discussions with various engineers and researchers about the damage to flexure-governed walls in Chile, two somewhat different interpretations tend to be offered, as summarized in Table 1. As shown in the table, the two interpretations lead to different conclusions about the cause of this damage and the 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.

Some evidence from the Chile earthquake points toward the ***buckling-first*** interpretation, in that:

- 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.
- All of the damaged walls reported had inadequate transverse reinforcement. 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 to see some damage even to well-detailed walls.

The *buckling-first* interpretation is also consistent with behavior observed in the test specimen by Thomsen and Wallace (2004), shown in Fig. 3a, which showed bar buckling occurring suddenly without significant prior spalling.

The ATC-94 project will analyze damaged and undamaged walls 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 are what lead to the damage. The implications of the *spalling-first* scenario would lead to requirements for a greater amount 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.

If the *buckling-first* interpretation is validated, the emphasis of design for flexural walls such as those in Chile should focus on restraining bars from buckling, because if this is done the questions of strain demand and strain capacity may be less critical.

Table 1 Summary of two possible interpretations of the flexural wall failures with longitudinal bar buckling that were observed in Chile.

Observation	*Spalling-first* interpretation	*Buckling-first* interpretation
Cause/ initiation of failure	The failures are associated with flexural *compression* and occur because compression strain demand exceeds compression strain capacity of the concrete. Strain capacity may be smaller than traditional assumptions.	Failures begin with buckling of longitudinal bars that occurs as a result of high *tensile* strain that stretches the bars prior to a reversal into compression that causes buckling.
	Bar buckling occurs after spalling and crushing, as a consequence of the flexural compression failure.	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.
	High axial load contributes to the failures.	A deep neutral axis depth (in both tension and compression) is a more important variable than high axial load.
Vertical concentration of damage	Strain demand is large because the walls show a very short plastic hinge zone (of high compression strain in the concrete).	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 continued 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 core area is small and the pattern of spalling indicates that plastic-hinge length is short. 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 can be restrained from buckling, the compression plastic hinge length can be longer, and the strain demands will not be so high. Thus performance would be improved, with little visible damage.

COMPONENT BEHAVIOR MODES

While post-earthquake seismic evaluation of building structures can involve complex considerations of site seismicity and soil characteristics, to estimate earthquake demands, studies (Zhang et al 2011) have shown that it is also possible, lacking an estimate of earthquake demands, to gain an important understanding of a building's response behavior through consideration of the relative strengths of the various structural components and actions. In its simplest application, this process involves evaluating the hierarchy of strength of the building's structural elements and identifying the governing mechanism of lateral deformation by hand calculation. FEMA 306 (ATC 1999) outlines a version of this process in which structural components are categorized according to their strength hierarchy and plastic mechanism (Fig. 4, Maffei et al 2000).

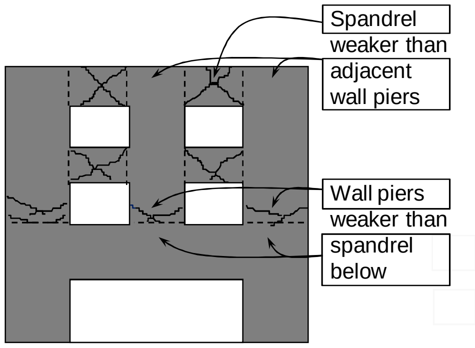


Fig. 4 Illustration from FEMA 306 of a method for determining the governing mechanism of non-linear lateral deformation in a concrete pier-spandrel system.

For example, the pier-spandrel system shown in Fig. 5a is a portion of a high rise building that suffered severe damage and partial collapse in the earthquake. Though not necessarily the cause of collapse, it is clear from the photo that wall piers suffered severe shear damage when constrained between deep spandrels, while the spandrels suffered flexural damage where they connect to heavier piers at the extreme bays. Based on this observation, engineering calculations of element strength should confirm that piers are shear-governed, and that the governing plastic mechanism for the system is as shown in Fig. 5c.

The structural behavior modes considered in this analysis for individual wall piers and spandrels were flexural yielding, sliding shear, and diagonal tension failure, computed based on strength equations from FEMA 306. FEMA 306 calculations for diagonal shear strength distinguish between the capacity corresponding to low and high displacement ductility demands on wall type elements. The geometry, reinforcement details, and specified material properties were obtained from available structural drawings for the building, while expected material properties for concrete and steel reinforcement were estimated based on PEER (2010). The axial load was calculated based on tributary area, self-weight, and typical dead and live uniform loads defined for a residential building by the NCh433Of.96 Chilean code (INN 1996). The resulting axial load ratio on the wall piers represents approximately 2% of their nominal axial capacity. Axial loads produced by seismic forces were disregarded in the simple hand calculations presented here. For each wall pier and spandrel beam, Table 2 presents the shear force corresponding to each of the behavior modes considered; the behavior mode with the lowest corresponding shear force is expected to govern the element's seismic response. These hand calculations confirm that the expected shear strength of individual piers is less than the shear that would correspond to flexural yielding.

In some buildings, pier-spandrel joint equilibrium can help to identify whether damage is expected to occur in piers or spandrels. However, for this example, a plastic mechanism analysis is necessary because the stronger end piers affect the overall mechanism of lateral deformation for the system.

For the plastic analysis of the four-story four-bay pier-spandrel system, several plastic mechanisms were considered, including shear failure of intermediate piers (Fig. 5c), spandrel shear failure between pier lines (Fig. 5d), and others. Virtual work principles and Lower Bound Theorem were applied to estimate the collapse load, assumed to be applied to the system at the top level of the subassembly. Indeed, the plastic mechanism shown in Fig. 5c corresponds to a lower plastic load than other possible mechanisms and is also consistent with the post-earthquake behavior observed.

Given that this behavior mode and mechanism are reasonably predictable by straightforward engineering calculations, the question could be considered whether design codes should require engineers to identify the governing behavior mode and mechanism, for example using capacity design principles. Currently US and Chilean codes make only a small distinction (via the strength reduction factor) in the assumed ductility capacity between shear versus flexural behavior. Shear failure of 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.

US and Chilean codes currently do not require investigation of the governing mechanism for pier and spandrel systems. While codes have “strong-column-weak-beam” provisions for moment frames, there are no such requirements for walls with openings to avoid strong-spandrel/weak-pier behavior, which can lead to a story mechanism in some wall structures. 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 the ATC-94 project, studies of lateral-force-resisting elements, such as cantilevered and coupled walls, will be performed to investigate how designers can best evaluate the expected behavior of these elements.

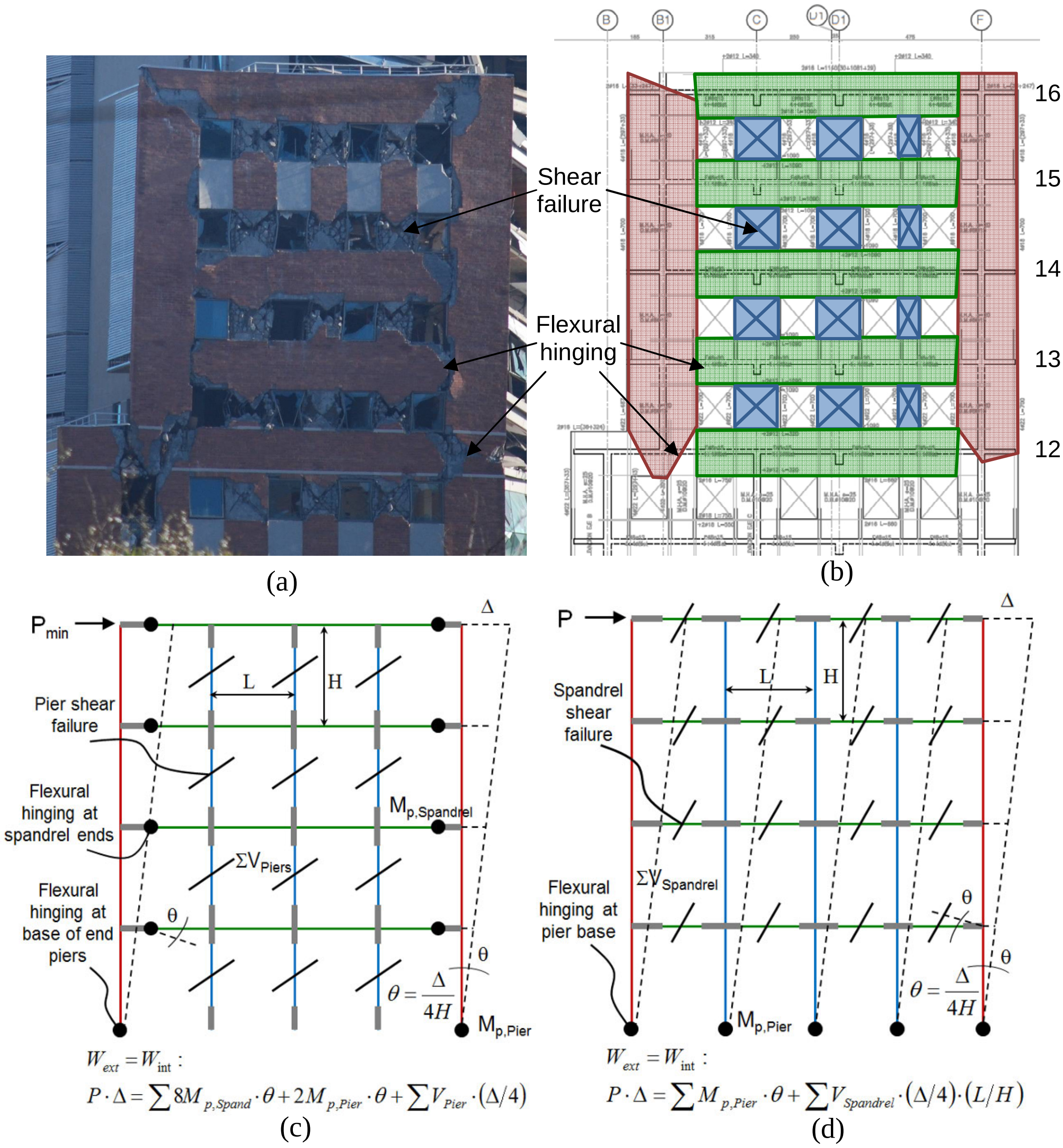


Fig. 5 Pier-spandrel system: (a) observed damage (photo from EERI team) (b) idealized structural elements (c) plastic mechanism observed and validated by calculation (d) example of a plastic mechanism considered and shown by calculation not to govern.

Table 2 Wall pier and spandrel beam strength and expected behavior mode

Element			Behavior modes considered, and corresponding shear force (kip)					
					Diagonal Tension			
Type	**Line**	**Level**	**Flexure**	**Sliding shear**	**ACI 318**	**FEMA 306, low ductility**	**FEMA 306, high ductility**	**Governing behavior, FEMA 306**
External Pier	B1,F	12	290	435	280	230	139	Diagonal Tension
		13	235	372	280	228	137	
		14	235	372	280	226	134	
		15	235	372	280	224	132	
Internal Pier	C	12	401	365	206	182	115	Diagonal Tension
		13	318	318	206	179	113	
		14	318	318	206	177	110	
		15	318	318	206	175	108	
Internal Pier	D1,E	12	304	298	168	146	92	Diagonal Tension
		13	239	292	168	144	90	
		14	239	292	168	143	88	
		15	239	292	168	141	87	
Spandrel	B1-F	13-16	236	155	199	144	76	Diag. Ten.

BUILDING BEHAVIOR

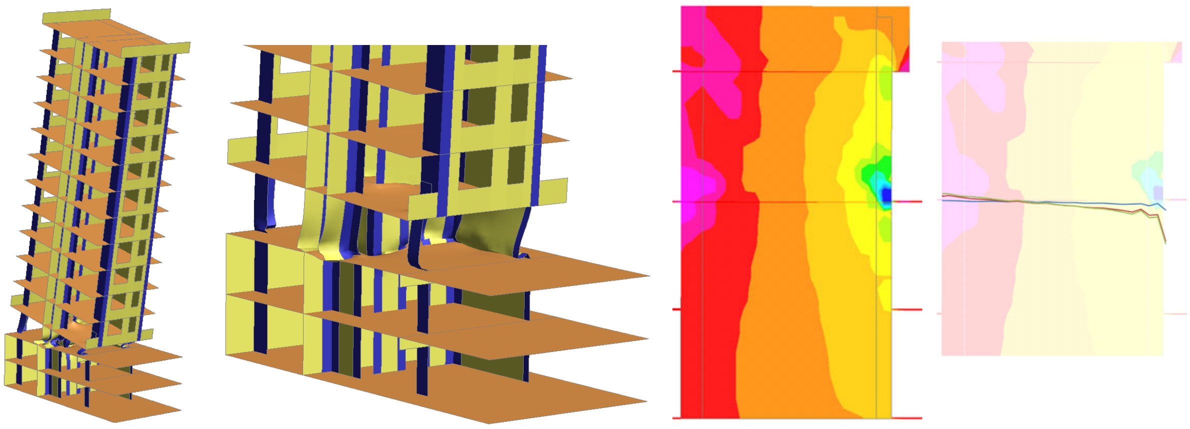
To complement studies of specific component behavior, whole building studies examine how structural elements interact with each other and how component behavior affects overall building performance. Working groups on the ATC-94 project are evaluate whether computer analysis models can accurately represent the location, type, and severity of damage observed in buildings that were affected by the earthquake. These studies include detailed non-linear dynamic finite element models as well as simpler models with tools that are more widely used by design practitioners. Objectives are to provide recommendations for what structural elements are important to consider in seismic analysis, such as coupling slabs or other elements not designated as primary lateral force-resisting elements, and for calibrating simpler models based on the findings from detailed models in this study.

Some good agreement has been found between results from sophisticated nonlinear finite element models and the apparent failure mechanisms observed in certain buildings. A preliminary study has been conducted using a three-dimensional finite element model of a representative slice of a concrete wall building affected by the earthquake, using the model shown in Fig. 6. The walls and floors of the building are represented by a detailed mesh of non-linear shell elements with an advanced constitutive formulation for reinforced concrete available in the LS-DYNA software. The shell elements have material zones defined through their thickness such that unconfined and confined concrete and the reinforcement planes are correctly positioned; this enables interaction of in-plane actions and out of plane bending (and wall buckling) to occur. This type of model does not require 'plane sections to remain plane' along a wall, and simulates the in-plane axial-flexure-shear interaction without empirical combination. LS-DYNA's large-deflection explicit solver enables softening behavior, buckling and incipient collapse to be captured without numerical convergence problems. The model captures the strain history of the wall longitudinal reinforcement and accounts for prior tension strain and Bauschinger softening that contributes to wall buckling or bar buckling. At this point the model does not address bar buckling in a fully explicit manner, but this capability could be added.

The 'slice' model (Fig. 6a) of the building was built based on the structural drawings. Few 'engineering' decisions and pre-calculations need to be made in assembling models of this type since all the walls, floors, reinforcement, openings, discontinuities etc. are represented explicitly. Almost all the concrete is unconfined since the reinforcement has few cross-ties and the bar spacings are large. The base of the model was subjected to the tri-directional motion time histories from the nearest recording station in Concepcion (Boroschek et al 2010).

Under seismic excitation this model predicts concrete crushing failure, initiated where concentration of compression strain occurs in the extreme fiber of the wall at ground level. The compression failure zone does not spread vertically, but propagates across the width of the wall in successive cycles, during which the sway period of the building also elongates. Eventually, as vertical carrying capacity is lost at both ends of the wall the building topples under gravity (Figs. 6a and 6b). Fig. 6c shows a close-up of the lower stories of the wall at the initiation of collapse, with a concentration of vertical compression strain occurring at the first floor level. As shown in Fig. 6d, the strain distribution along this section of the wall is not linear in the compression zone; compression strains increase sharply near the extreme fiber. This preliminary analysis supports the hypothesis that compression failure in wall boundary elements can lead to collapse in certain cases.

The concentration of strain at extreme fibers is observed in laboratory tests on walls, but is not predicted by a traditional analysis that assumes that plane sections remain plane. The observation of compression damage over only a short height of the wall is also predictable for ‘strain softening’ failure associated with unconfined concrete and low reinforcement ratio. Further studies will be directed at refining models to determine what types of conditions are most vulnerable and what adjustments can be made to practical analysis methods and design practices to account for this behavior. Possible recommendations could include avoiding vertical discontinuities in compression critical areas of wall, adjusting analysis assumptions or limits on concrete compression strain, or using special detailing requirements to improve compression and cyclic behavior of wall boundaries.



(a) (b) (c) (d)

Fig. 6 Analysis of wall with vertical discontinuity (a) model deformed shape at failure (b) close-up of lower stories (c) vertical strain concentration at ground floor (c) strain distribution on wall section

CODE IMPLICATIONS

Chilean engineers have proposed several changes to building codes to address behaviors observed in the earthquake. The following is a partial list of proposals that have been considered (Bonelli et al 2012):

- Apply a correction factor to increase the design displacement $_u$ used to determine confinement requirements for walls.
- Consider longitudinal reinforcement in the entire flange width when determining the strength/behavior of flanged walls under flexure and axial loads.
- Limit the longitudinal bar diameter to 1/9 of the least dimension of the wall thickness.
- Require that the bar diameter of transverse reinforcement be at least 1/3 the diameter of the confined longitudinal bar.
- Require that transverse reinforcement be anchored with standard hooks to extreme longitudinal bars in the wall.

- Limit extreme compressive strain to 0.008 under design displacement $_{u}$. (Or, more directly in line with principles of capacity design, require that extreme steel tension strain be at least 0.004 when concrete reaches assumed strain of 0.003.)
- Require a minimum wall thickness of 1/25 the unsupported wall height; if less than 1/16 the unsupported height, then out-of-plane buckling must be studied. (Or, more conservatively, require a minimum wall thickness of 1/16 the unsupported wall height.)
- Apply a shear amplification factor of 1.4 to the code-prescribed earthquake loads for designing wall web transverse reinforcement, unless principles of capacity design are used to protect against shear failure.
- For sections with large longitudinal reinforcement ratios or limited cover, require that the total transverse reinforcement area in the lap length to be equal or greater than the area of the lapped bar.

The ATC-94 project will also be considering areas 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.

CONCLUSIONS

Damage to mid-rise and high-rise concrete wall buildings caused by the 2010 Chile Earthquake offers a rare and valuable opportunity to study buildings in detail to gain practical lessons for structural design. Observed damage such as concrete crushing and reinforcing bar buckling in wall boundary elements could be interpreted in different ways with different implications on design. Overall wall buckling, pier-spandrel wall behavior, and damage from discontinuities and configuration effects are also key issues. Sophisticated three-dimensional non-linear finite element models of Chilean buildings have been built and initial response-history analysis results show good agreement with damage observations. Through the ATC-94 project, a team of researchers and practitioners is developing recommendations for modifying design practices based on studies of damaged Chilean wall buildings, leading to efforts to address undesirable behavior modes and improve the seismic performance of concrete wall buildings.

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REFERENCES

American Concrete Institute (1995). *Building Code Requirements for Structural Concrete (ACI 318-95) and Commentary*. Farmington Hills, Michigan, USA.

American Concrete Institute (2008). *Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary*. Farmington Hills, Michigan, USA.

American Society of Civil Engineers (2010). *ASCE 7-10: Minimum Design Loads for Buildings and Other Structures*. Reston, Virginia, USA.

Applied Technology Council (1999). *FEMA 306: Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings*. Federal Emergency Management Agency, Washington, DC, USA.

Bonelli, P., Restrepo, J.I., and Boroschek, R. (2012). "The 2010 Great Chile Earthquake – Changes to Design Codes." *Proc. One Year after the 2011 Great East Japan Earthquake*, Tokyo, February.

Boroschek, R., Soto, P., Leon, R. (2010). *Maule Region Earthquake, February 27, 2010, Mw 8.8.* University of Chile Civil Engineering Department, RENADIC Report 10/08, Rev. 2. Ground motions accessed 4 Oct. 2011 from <http://terremotos.ing.uchile.cl/>

Cowan, H., Beattie, G., Hill, K., Evans, N., McGhie, C., Gibson, G.., Lawrence, G., Hamilton, J., Allan, P., Bryant, M., Davis, M., Hyland, C., Oyarzo-Vera, C., Quintana-Gallo, P., and Smith, P. (2011). "The M8.8 Chile Earthquake, 27 February 2010." *New Zealand Society of Earthquake Engineering Bulletin*, Vol. 44, No. 3, September.

Earthquake Engineering Research Institute (2010). "The Mw 8.8 Chile Earthquake of February 27, 2010." *EERI Special Earthquake Report*, June.

Instituto Nacional de Normalizacion (1996). *NCh433.Of96: Earthquake resistant design of buildings, Official Chilean Standard.*

Maffei, J., Comartin, C., Kehoe, B., Kingsley, G., and Lizundia, B. (2000). "Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings." *Earthquake Spectra*, Vol. 16, Issue 1, 263-283.

Moehle, J. (2010). "The 27 February 2010 Chile Earthquake: Implications for U.S. Building Codes and Standards, Detailed Recommendations from 2 June 2010 Meeting." Meeting organized by American Society of Civil Engineers, National Institute of Technology, and Pacific Earthquake Engineering Research Center, San Francisco, California, USA.

Pacific Earthquake Engineering Research Center (2010). *Guidelines for Performance-Based Seismic Design of Tall Buildings*. PEER Center Tall Buildings Initiative, Berkeley, California, USA.

Thomsen IV, J.H. and Wallace, J.W. (2004). "Displacement-Based Design of Reinforced Concrete Structural Walls—Experimental Verification." *Journal of Structural Engineering*, American Society of Civil Engineers (ASCE), Vol. 130, No. 4, 618-630.

Zhang, D., Federico, G., Telleen, K., Schellenberg, A., Fleischman, R., and Maffei, J. (2011). "Structural Analysis to Replicate Observed Damage to Engineered Buildings from the January 2010 Haiti Earthquake." *Proc. 2011 ASCE Structures Congress*.