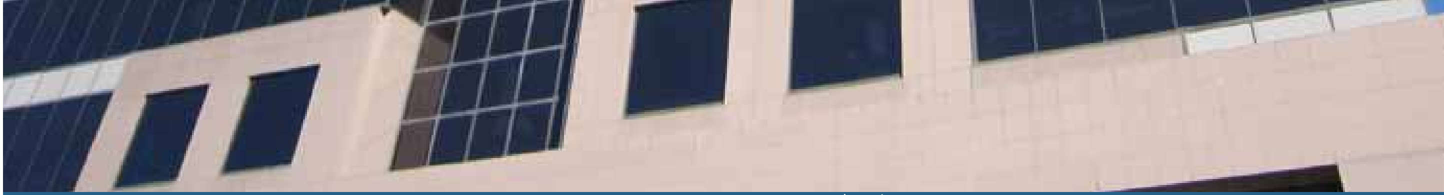

Photo: Joseph R. Maffei

PCI Reconnaissance Team Report

Observations from the February 27, 2010, earthquake in Chile

S. K. Ghosh and Ned Cleland

- The authors were part of a PCI reconnaissance team of investigators who went to various locations affected by the February 2010 earthquake in Chile.
- The 1996 Chilean seismic code was similar to the then-current UBC and ACI 318 codes, except that boundary elements and special transverse reinforcement were not required in structural walls. This exception was revoked in the 2009 version of the Chilean code due to a trend towards thinner walls.
- The number of deaths and the amount of property loss were not disproportionate to the severity of the earthquake. Much of this is attributable to Chile's history of adoption and implementation of adequate building codes.
- The 2010 emergency changes to the Chilean Building Code have far-reaching implications for the special structural wall design provisions in ACI 318.

The earthquake that shook Chile at 3:34 a.m. on Saturday, February 27, 2010, was one of the most devastating in the history of the country, which has a 2650 mi (4270 km) coastline along the Pacific Ring of Fire. The moment magnitude issued by the U.S. Geological Survey[1] was 8.8. The earthquake was followed by hundreds of aftershocks, the strongest measuring from 6.0 to 6.9 on the moment magnitude scale. **Table 1** gives the details of the earthquake.[1]

Maximum ground acceleration of up to 0.65g was recorded at Concepción, and more than 6.6 ft (2 m) of uplift was observed near Arauco on the coast.[1]

The earthquake was generated at the gently sloping fault along which the Nazca plate moves eastward and downward beneath the South American plate (**Fig. 1**). The two plates are converging at 2¾ in. (70 mm) per year. The fault rupture, largely offshore, exceeded 60 mi (100 km) in width and extended nearly 300 mi (500 km) parallel to the coast.

A comprehensive written record beginning in the mid-16th century describes large damaging earthquakes throughout the region that was affected by the February 27, 2010, earthquake. An 1835 M8.2 (M = moment magnitude)

Concepción earthquake is notable because famed naturalist Charles Darwin and naval officer Robert FitzRoy provided observations and comments.[1] Since the beginning of the 20th century, there have been M8.2 earthquakes in 1906, 1943, and 1960, and an M8.0 earthquake in 1985.[1] The 1960 M8.2 earthquake was a foreshock that occurred a day before the great M9.5 Chilean earthquake of 1960.[1]

The 2010 earthquake that is the subject of this paper struck in an area previously identified as a seismic gap extending from Constitución in the north to Concepción in the south with a projected worst-case potential to produce an earthquake with M between 8.0 and 8.5.[2] The rupture extended beyond the northern and southern boundaries of the gap, overlapping extensive zones already ruptured in 1985 and 1960.[3]

Table 1. Earthquake details

Moment magnitude	8.8
Date-time	Saturday, February 27, 2010, at 03:34:14 a.m. at epicenter
Location	35.909°S, 72.733°W
Depth	21.7 mi
Region	Offshore Maule, Chile
Distances	60 mi NW of Chillán, Chile 65 mi NNE of Concepción, Chile 70 mi WSW of Talca, Chile 210 mi SW of Santiago, Chile

Source: U.S. Geological Survey. Note: 1 mi = 1.61 km.

Strong-motion records

The University of Chile's strong-motion instrumentation array recorded motions at several sites in the heavily stricken region. Some of the digital data have been processed and reported by the University of Chile[4] and by Boroschek et al.[5] **Figure 2** shows three sets of recorded accelerograms and the corresponding response spectra from the Santiago area. Ground accelerations exceeding 0.05*g* lasted more than 60 sec according to most of the records. Elastic response spectra of several records are higher than the elastic design spectrum of the Chilean seismic code, NCh433-2009.[6]

Figure 3 shows horizontal ground motion accelerograms from downtown Concepción. A special characteristic of the records is the long duration of strong shaking (90 sec or more). Also shown are the acceleration and displacement response spectra for the same ground motions. The acceleration spectra show unusual second peaks at periods of 1.5 sec and longer. The acceleration and displacement spectra are compared with the design spectra for soil types II, III, and IV as defined in NCh433-2009.

Figure 4 shows horizontal ground motion accelerograms from Colegio San Pedro, across the Bio Bio River southwest of downtown Concepción, along with their acceleration and displacement response spectra. The acceleration spectra show second peaks at periods of about 0.8 sec. Again, the design spectra for soil types II, III, and IV are also shown.

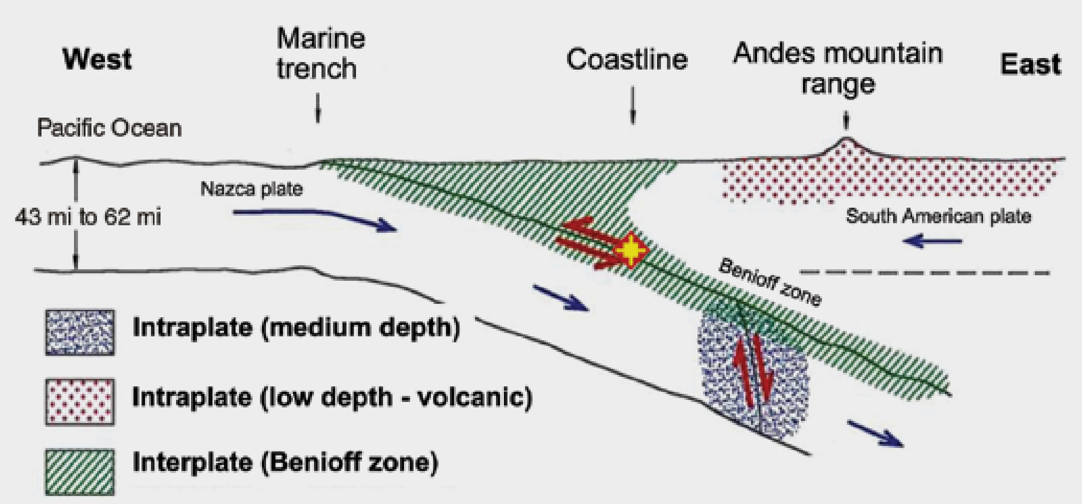

Figure 1. The source of the Chile earthquake is at the convergence of the Nazca and the South American plates. Source: Roberto Leon presentation at www.eqclearinghouse.org/20100227-chile/wp-content/uploads/2010/04/Leon-Chile-Earthquake.pdf. Note: 1 mi = 1.61 km.

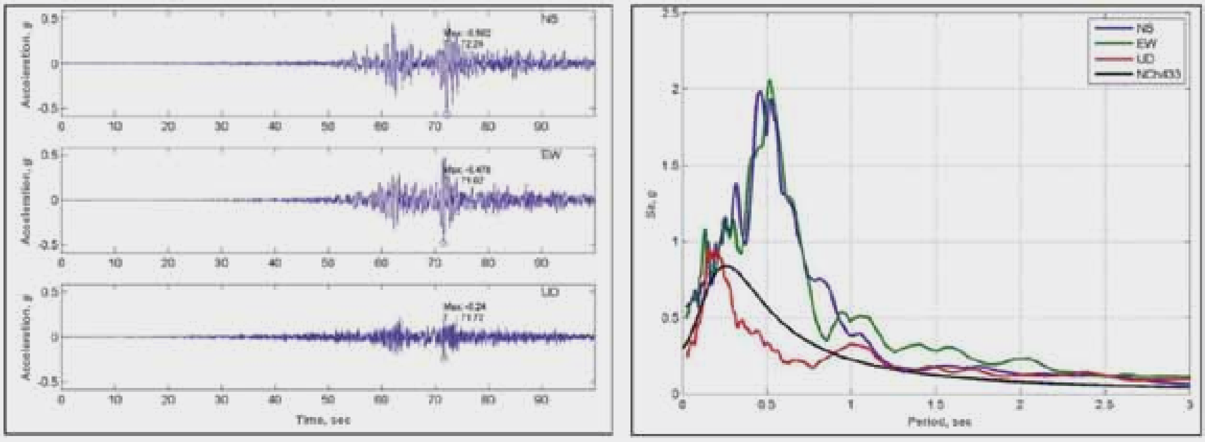

Figure 2. Accelerograms and corresponding acceleration response spectra (β = 5%) from the Santiago, Chile, area. Source: Boroschek et al. 2010. Note: g = acceleration due to earth's gravity; EW = east-west; NS = north-south; S_a = spectral acceleration; T = time; UD = up-down; β = damping coefficient.

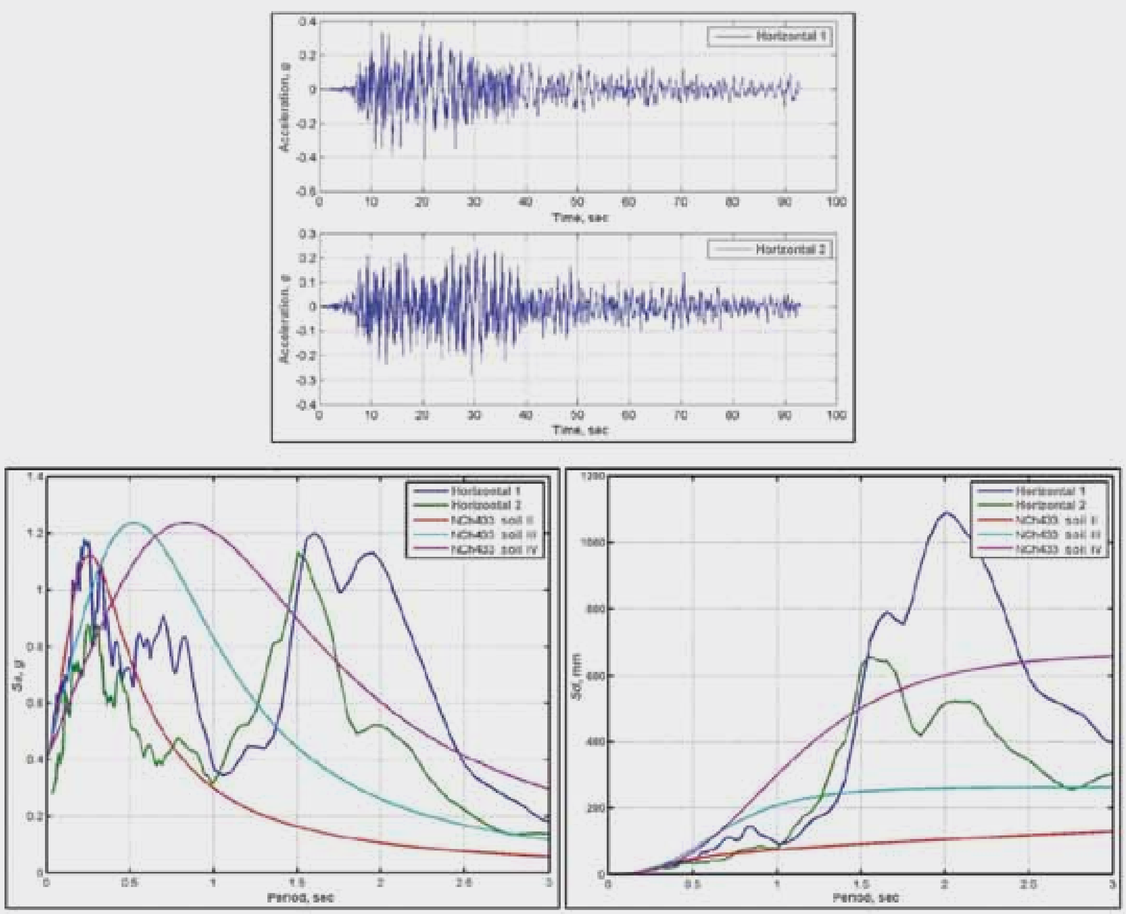

Figure 3. Accelerograms and corresponding acceleration and displacement response spectra (β = 5%) from downtown Concepción, Chile. Source: Boroschek et al. 2010. Notes: g = acceleration due to earth's gravity; S_a = spectral acceleration; S_d = spectral displacement; β = damping coefficient. 1 cm = 0.4 in.

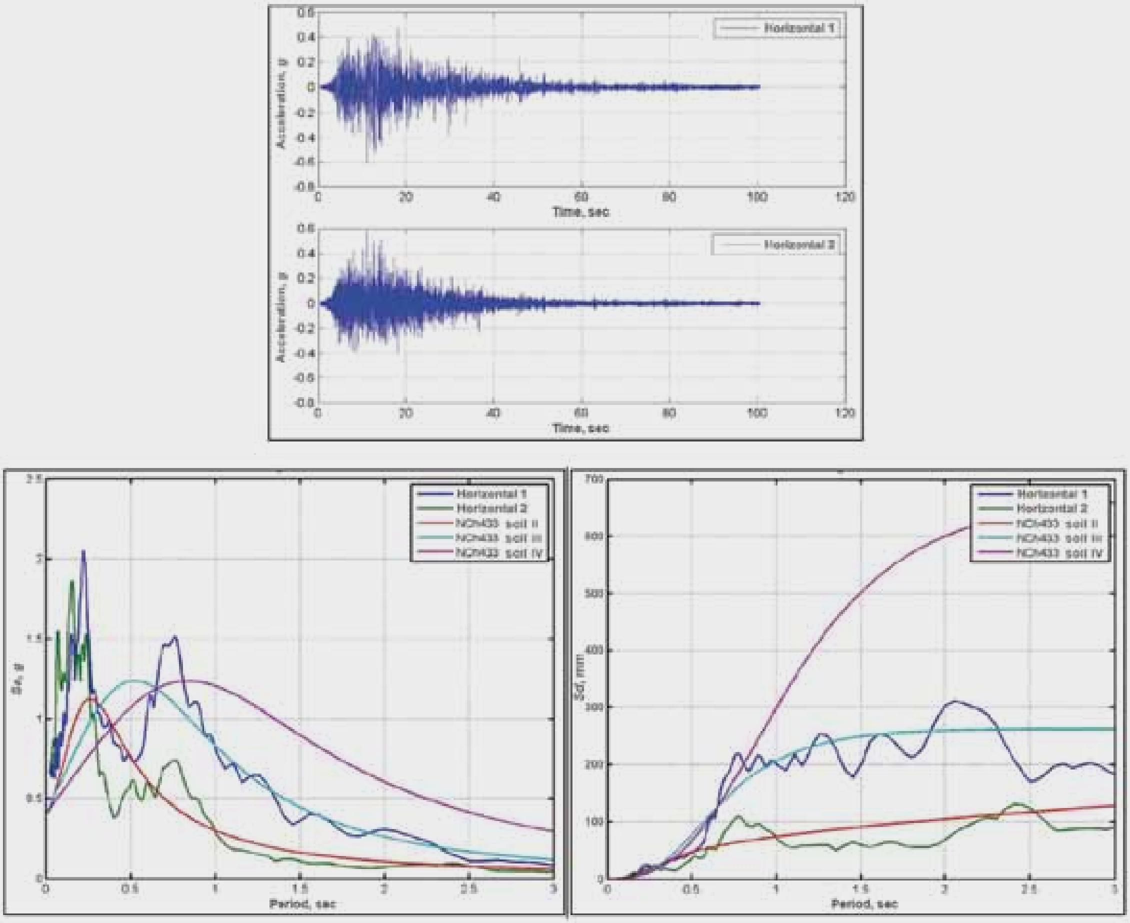

Figure 4. Accelerograms and corresponding acceleration and displacement response spectra (β = 5%) from Colegio San Pedro, Chile. Source: Boroschek et al. 2010. Notes: g = acceleration due to earth's gravity; S_a = spectral acceleration; S_d = spectral displacement; β = damping coefficient. 1 cm = 0.4 in.

PCI investigation

The entire team visited Chile April 26–29, 2010, to investigate damage in Santiago, Concepción, Talca, Chillán, Coronel, and Chillán Viejo. Three team members spent an additional day visiting Valparaiso/Viña del Mar.

Other investigations

A team organized by the Earthquake Engineering Research Institute (EERI) investigated the effects of the Chile earthquake. The team was assisted by local university faculty and students. Geotechnical Extreme Events Reconnaissance (GEER) contributed geosciences, geology, and geotechnical engineering findings. The Technical Council on Lifeline Earthquake Engineering (TCLEE) contributed a report based on its reconnaissance. Based on its own investigation and the GEER and TCLEE input, EERI published the *EERI Special Earthquake Report—June 2010* as an insert in EERI's monthly newsletter.[3] EERI also set up a Chile Earthquake Clearinghouse.[7]

The Structural Engineering Institute of the American Society of Civil Engineers (ASCE) sent an earthquake assessment team to assess the effectiveness of Chile's building methods and codes, which closely parallel those used in the United States. The primary purpose of the team was to determine whether changes are warranted to the U.S. codes, standards, or practice in general and to ASCE 7-10 *Minimum Design Loads for Buildings and Other Structures*[8] and ASCE 41-06 *Seismic Rehabilitation of Existing Buildings*[9] in particular. Part of the team traveled to different locations to study structures built after the 1985 earthquake, when more-detailed building codes were implemented. The remainder of the team focused on structures such as steel mills and power plants.

The report of the assessment team has not been published yet, so no definite conclusions are available. The team observed several differences between the Chilean standards and those in the United States. For example, the walls of buildings are much thinner than is required in the United States and do not contain as much reinforcement. Despite its observations of significant nonstructural damage and their review of plans, the team did not identify anything

Figure 5. Damaged coupling beam in Viña del Mar, Chile.

that would necessitate substantive changes to U.S. standards such as ASCE 7-10[8] or ASCE 41-06.[9]

The Los Angeles Tall Buildings Structural Design Council also sent a team. Its report is not yet available; however, a presentation is posted on its website.[10]

With so much information already available or coming soon, this report concentrates largely on the performance of precast concrete structures, though some other aspects are also included.

Building performance

Mid- to high-rise buildings in Chile are predominantly of reinforced concrete construction. Most of these buildings use structural walls to resist both gravity loads and earthquake forces. Dual systems of walls and frames are occasionally used in newer construction. Typical wall cross-sectional area–to–floor area ratios are high compared with values commonly used in U.S. concrete building construction.

In 1996, the Chilean seismic code (NCh433-1996)[11] adopted analysis procedures similar to those in the 1997 *Uniform Building Code* (UBC).[12] However, there are no prohibitions or penalties related to vertical or horizontal system irregularities. NCh433-1996 also enforces provisions of *Building Code Requirements for Structural Concrete (ACI 318-95) and Commentary (ACI 318R-95)*,[13] with one significant exception, as noted in the following paragraph.

Having observed and investigated the performance of reinforced concrete buildings during the 1985 Chile earthquake, Wood[14] and Wallace and Moehle[15] reached nearly identical conclusions. The primary variables that determine the need for confined boundary elements in shear walls were found to be the ratio of wall cross-sectional area to floor-plan area, the wall aspect ratio and configuration, the axial load on the wall, and the reinforcement ratio of the wall. Wallace and Moehle concluded that concrete confinement in walls of bearing-wall buildings may be necessary at the extremities of walls having T-shaped, L-shaped, or similar cross sections, but that confinement is typically not required for symmetrically reinforced, rectangular wall cross sections. Wallace and Moehle[15] went on to state, "The good performance of the majority of these [bearing wall] buildings during the March 3, 1985, earthquake suggests that bearing walls with limited detailing may be an effective construction form for earthquake resistance. Although buildings in Chile are designed for roughly the same lateral forces as those in regions of high seismic risk in the United States, the typical structural wall in a Chilean building does not require boundary elements or special transverse reinforcement."

Based on this, NCh433-1996[11] contained clause B.2.2, which states, "When designing reinforced concrete walls, it is not necessary to meet the provisions of paragraphs 21.6.6.1 through 21.6.6.4 of the ACI 318-95[13] code." These ACI 318-95 sections are for specially confined boundary elements at the edges of shear walls. NCh433-2009[6] rescinded this exception before the February 2010 earthquake because of a trend to use thinner walls more in recent years than in the past.

Tall concrete buildings are typically found in the metropolitan areas around Santiago, Valparaiso/Viña del Mar, and Concepción. In Viña del Mar, a number of buildings that were damaged in the 1985 earthquake and repaired suffered significant damage once again. However, damage was largely concentrated in newer buildings. The failure of one tall building in Viña del Mar was due to the wide spacing of transverse reinforcement in shear walls, which caused the vertical bars to buckle, in this particular case, without fracture. In many other cases the vertical bars did fracture.

Coupling beams over doorways typically have inadequate reinforcement. Many of these beams suffered damage (**Fig. 5**). Some buildings lacked coupling beams. In many of those cases, damage resulted from the slab acting as the coupling element. There were several instances of doors that jammed because of displacements in the walls on either side. Spalled cover on top of lap splices of wall boundary reinforcement was a common occurrence.

Four mid- to high-rise concrete buildings collapsed completely or partially. Two of these were nearly identical, side-by-side, five-story buildings in Maipú, Santiago (not visited by the PCI team). According to the *EERI Special Earthquake Report*,[3] these buildings had four stories of condominium units atop a first-story parking level with an irregular wall layout. Wall failures apparently contributed to the collapses.

A third collapsed building was the 15-story Alto Río condominium in Concepción (**Fig. 6**). The team was unable to examine closely the side of the building toward which it collapsed. According to the *EERI Special Earthquake Report*,[3] the structural drawings indicated that concrete walls on the facade were discontinuous and that the wall lengths were decreased in the first story on the side toward which the building collapsed. There was ample indication that the building had rotated about its corridor walls as it collapsed, leading to tension failures of the transverse walls on the side from which the photo was taken. Some of the wall vertical reinforcement fractured, and some lap splices failed on the tension side.

The fairly new 23-story O'Higgins office building in Concepción suffered partial story collapses at levels 10, 14, and 18, each coincident with a framing setback (**Fig. 7**). The perforated shear walls on the east face (shown) and south face showed damage to both wall piers and spandrels. The exterior north and west faces appeared undamaged.

The following observations of building performance emerged:

- **Axial stress in shear walls.** As mentioned previously, Chilean buildings typically contain many shear walls. This contributed to their relatively good performance during the 1985 earthquake. Newer buildings appear to have the same shear wall area in terms of the percentage of floor area, but many are significantly taller than before. This suggests that the axial stress in the walls of newer buildings is significantly higher than in older buildings. This may, at least in part, be responsible for the widely observed localized wall damage characterized by buckling of vertical reinforcement.

- **Confinement of wall boundary elements.** The exception made in NCh433-1996[6] to the specially confined boundary zone requirements of ACI 318-95[13] was explained previously. Considerable damage was observed in many wall boundary elements, including crushing of concrete and buckling and fracture of longitudinal reinforcement. The exception was rescinded in NCh433-2009. However, the ACI 318-08[16] requirements have now come into question. The trigger for requiring specially confined boundary zones should be reexamined. The reduction of the boundary element confinement requirements when specially confined boundary zones are not triggered should also be reviewed.

- **Vertical wall reinforcement.** This item has been described by Wallace:[17] "Many damaged walls were lightly reinforced and had unconfined lap splices. These walls were observed to have problems at lap splices or to suffer tension failures (or fractures during buckling following tensile elongation). Due to the long duration of the earthquake, the walls likely underwent a large number of cycles of loading. The possibility of a failure mode consisting of progressive concrete crushing and buckling or fracture of reinforcement across entire wall (unzipping) should be investigated."

Figure 6. The 15-story Alto Río Condominium in Concepción, Chile.

Precast concrete buildings

The precast concrete construction market in Chile does not include parking structures but does include bridges, office buildings, stadiums, warehouses, and industrial buildings. Some systems did not fare well during the February 27, 2010, earthquake. Many buildings of more recent construction did well, and some advanced precast concrete concepts proved their merit.

Gable frames

One precast concrete system that did not perform well was a precast concrete gable frame system at Parque Industrial Escuadrón. The roof of the single-story San José Fishery was formed by a series of these frames. The structure was reported to be 23 years old but appeared older. **Figure 8** shows the portion of the building still standing after the earthquake.

Figure 7. The 23-story O'Higgins office building in Concepción, Chile.

Figure 8. Precast concrete gable framed San José Fishery after the earthquake.

The gable frames were assembled using three standard parts, which include end columns, interior columns, and drop-in gables (**Fig. 9**). The columns have monolithic knee joints and include parts of the sloping gable members.

The roof was constructed over spaced precast concrete purlins that spanned between rows of these frames. The gable frame construction was similar to precast concrete frames that performed poorly in the 1999 earthquake near Izmit, Turkey. There are a few notable differences. In Turkey, the frame across the top of the column was a separate precast concrete element spliced to the column and not cast monolithically. The connections at the drop-in gables in the Turkish frames were pinned. The connections observed at the Chilean fishery were welded and apparently intended to provide strong connections for continuity. Some welded connections failed by fracture of the reinforcement welded to the embedded parts for lap and development with the precast concrete frame reinforcement. It is likely that the bars were not weldable. Another difference was that the Turkish precast concrete frames lacked any lateral bracing perpendicular to the plane of the gable frame. At the Chilean fishery, there were precast concrete diagonal braces in the end bays that remained standing. It is unclear whether there was additional diagonal bracing in the collapsed bays, but it was reported that the collapse started in one corner and progressed across the building to the braced bays still standing. The system of spaced purlins and light corrugated roof infill did not form a continuous diaphragm.

Column base failures revealed base anchor bars lapped with column bars lacking standard hooks and a lack of confinement reinforcement. The remaining debris also showed that some sections were hollow, formed with expanded polystyrene cores. These gable frames lacked the strength and ductility of special moment frames suitable for high seismic application.

It did not appear that this framing system was in common use. The PCI team did not find any examples of this framing of more recent vintage than at the San José Fishery.

Precast and cast-in-place concrete shear walls

Based on the examples of precast concrete construction that the PCI team was able to find, the industrial buildings that were constructed with precast concrete often included precast concrete walls as cladding. Unlike the practice in the United States, however, walls were not often used as the primary lateral-force-resisting system (LFRS). Examples where walls provided the lateral bracing were found in a pair of warehouse buildings that were constructed with long-span gable beams on columns and clad with double-tee walls.

The bays were 39 ft (12 m) wide and 66 ft (20 m) long. Gable-shaped girders spanned the 66 ft, and spaced precast concrete purlins spanned the 39 ft. The ends of the girders were fixed to the tops of the columns, but the purlins were pinned to the girders. The purlins were not continuous. At the outside edges, two-stemmed channels spanned between the girder-column frames, providing a stiff lateral support to the exterior cladding made with precast concrete double-tee walls. **Figure 10** shows a building portion with this framing. At the front of the building, the tee stems were turned out, and at the rear they were turned in. Concrete planks spanned

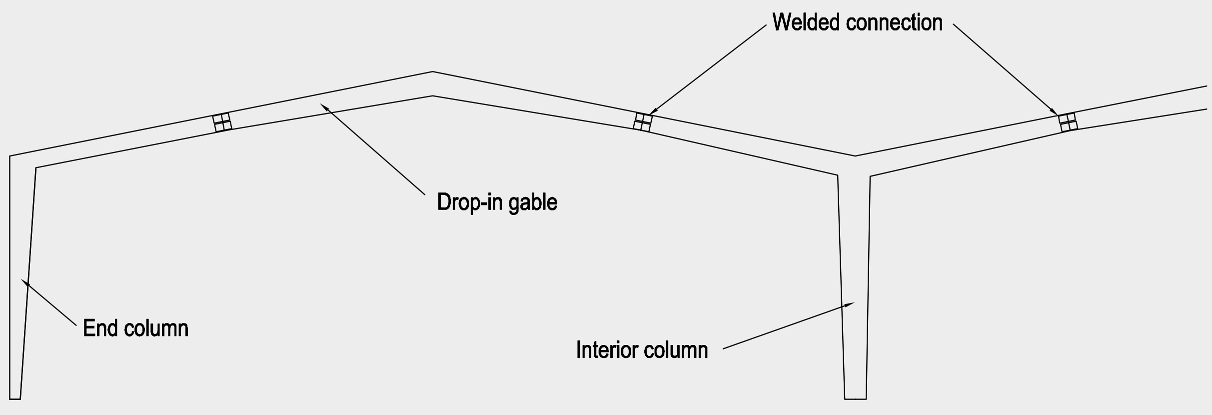

Figure 9. Gable frame system.

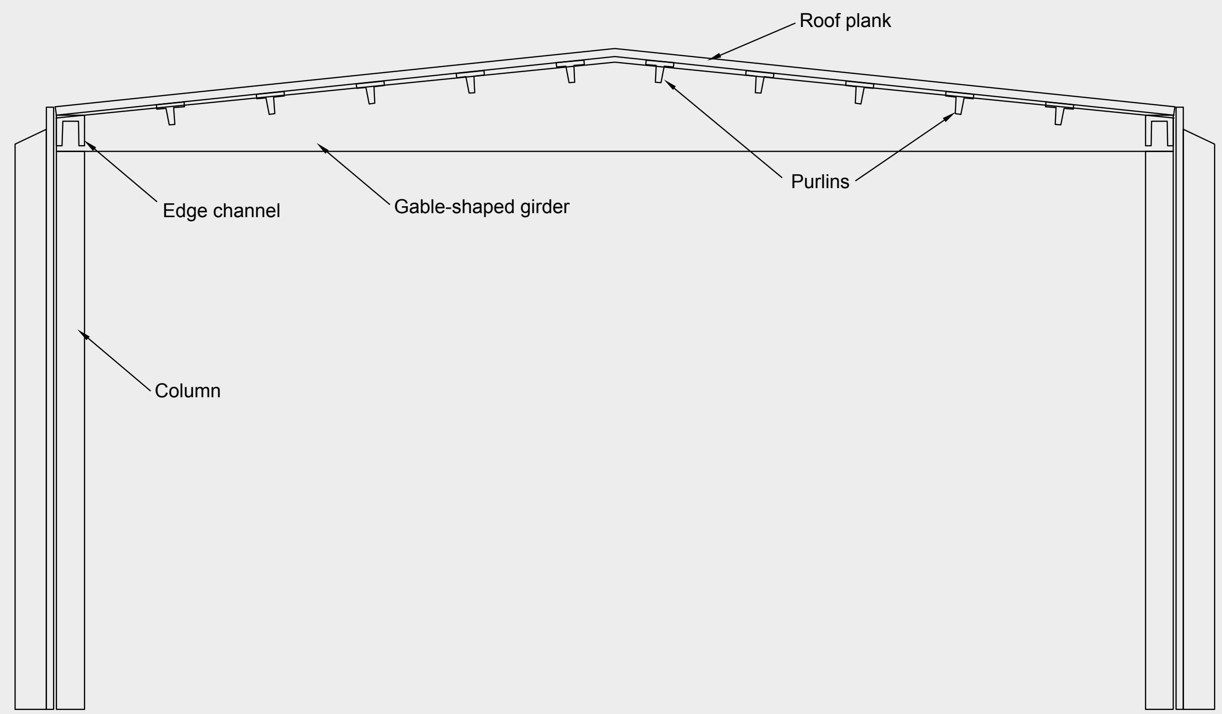

Figure 10. Framing system that uses long-span gable beams on columns and is clad with double-tee walls in an industrial building with offices.

over the purlins, but there did not appear to be connections between planks to form a diaphragm. There was no diagonal bracing in the plane of the roof deck. The wall cladding was connected to the precast concrete roof using long threaded rods that connected to the channels on the sides and the girders at the ends. These connections appeared to provide out-of-plane resistance but not a load path to transfer lateral forces into the plane of the wall cladding.

The shear walls for these buildings were isolated cast-in-place concrete walls in two bays on each side. These bays were the last short bay at one end and the second bay from the other end, just inside the two-level bay with offices at the front of the building. There were interior walls across the building at the first bay that included two floors of office space, but this did not provide effective bracing to the framing at the distant end. The column-to-girder connections apparently provided sufficient continuity for frame behavior in the direction of the frame. **Figure 11** shows a sketch of the building plan.

The team learned from the owner's representative that one of the buildings had soil saturation problems during construction that required soil improvement to a depth of

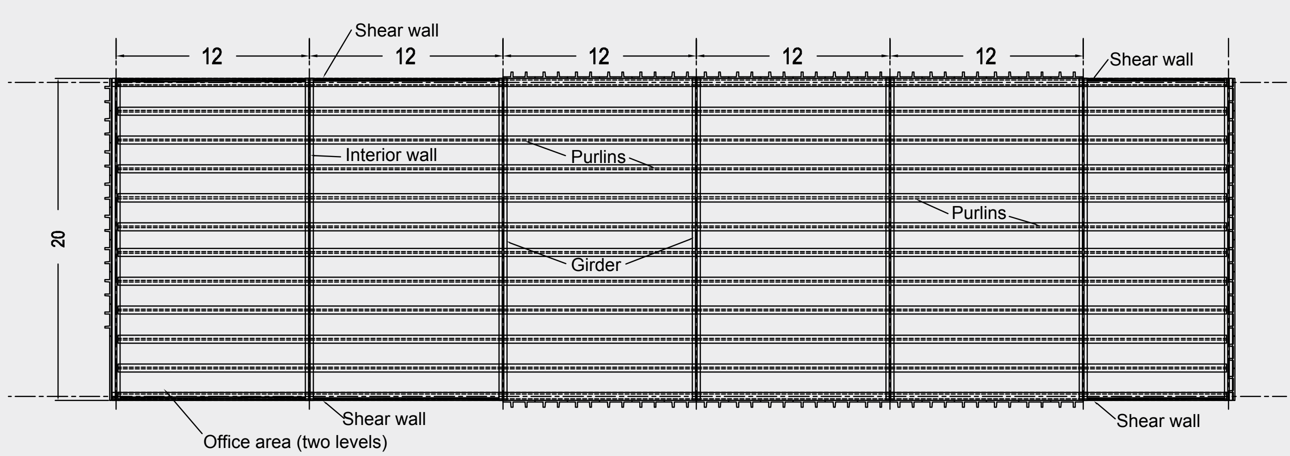

Figure 11. Roof framing plan used in an industrial building with offices. Note: All measurements are in meters. 1 m = 3.28 ft.

Figure 12. Front elevation of an office and warehouse building showing failed beam-column framing.

Figure 14. Reinforced concrete column with bar buckling and lack of confinement.

Figure 13. Rear elevation of the same office and warehouse building showing fallen exterior walls and exposed transverse walls and end columns.

Figure 15. Bent and broken connection angle that failed to hold walls to edge beam at roof.

4.6 ft (1.4 m). The other did not. The structure without soil improvement dropped the precast concrete purlins and roof in the two bays with cast-in-place concrete shear walls. The other building did not suffer damage. It appears that the lateral bowing in the roof girders caused a failure at the pins and loss of bearing for the purlins.

Although many precast concrete buildings constructed using shear walls have performed well in past earthquakes, the LFRS requires a complete load path that ties all components together. In this case, it appears that the roof framing with spaced purlins and without connections between roof planks lacked a diaphragm. Failures occurred at the pinned purlin bearings where movement between the roof and the supporting girders was not sufficiently restrained.

Another combined office and warehouse consisting of a precast concrete building with transverse and longitudinal walls suffered major local failures and partial collapse. The structure had two-level beam and column framing on the front, with column spacing at about 20 ft (6 m) and stairways every 80 ft (24 m). Transverse walls separating the units were spaced at 40 ft (12 m), and the length from front to back was from 66 ft to 82 ft (20 m and 25 m). The rear was enclosed by vertical precast concrete walls with loading docks and doors. **Figures 12** and **13** show photos of the collapse.

Although there appeared to be an ample number of walls to provide lateral bracing for the structure, failures likely occurred because of inadequate connections and a lack of seismic detailing for strength and ductility. Most of the walls that clad the exterior of the stairs on the front elevation fell away from the structure because of out-of-plane forces that caused the connections to fail. Failed gravity columns showed bar bucking and a lack of confinement (**Fig. 14**).

On the rear elevation, the tops of the walls were connected to the spandrel girders at the roof through a thin angle that

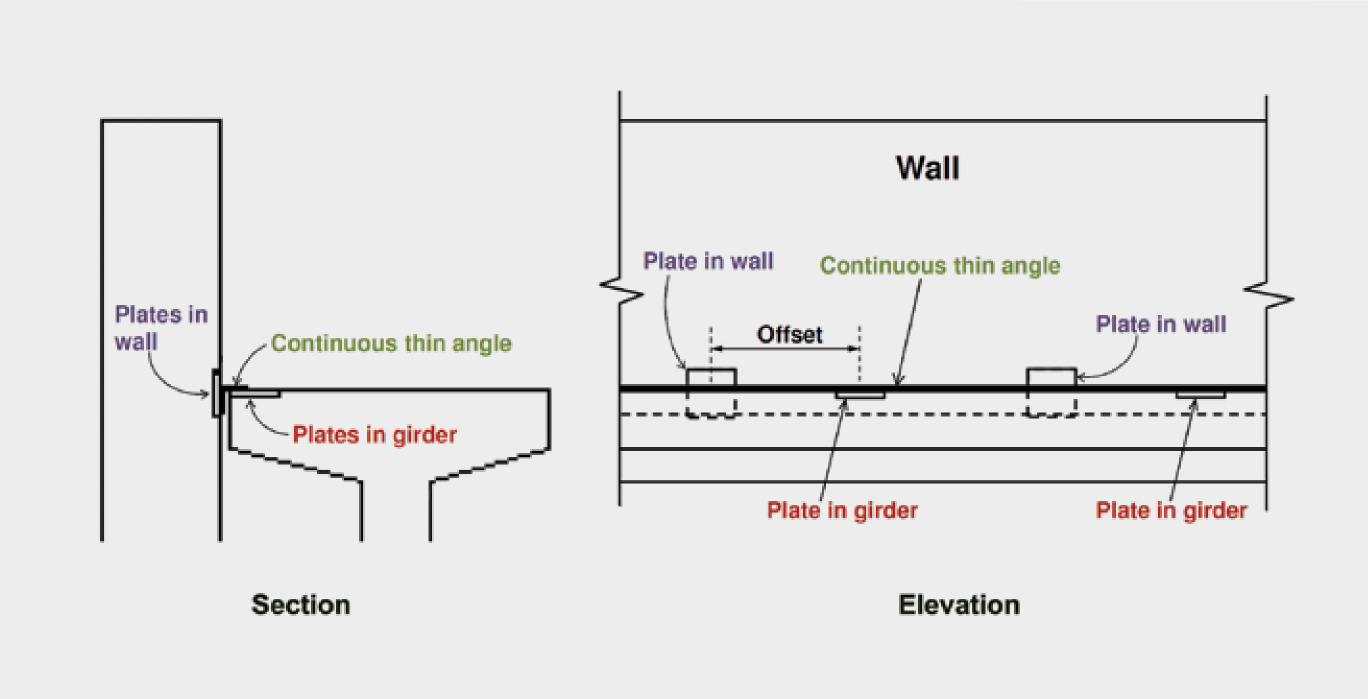

Figure 16. Misaligned wall-girder connection that failed.

spanned between embedments in the walls and girders. **Figure 15** shows a remnant of bent and broken angle still welded to embedments in the back of the fallen spandrel panel. The welds to the plates embedded in the spandrel girder were torn loose. **Figure 16** shows that the continuous angle allowed the welds to be made even if the plates in the walls did not align with the plates in the spandrel girder. The out-of-plane forces, however, caused bending in the angle and prying on the welds. There were also locations that appeared to have field-installed anchor rods in the walls that projected into the cast-in-place concrete topping over the roof. Walls had shallow breakout cones that appeared to correspond to short lengths of bent dowels projecting from the edge of the topping. In addition, the walls were relatively thin, about 6 in. (150 mm), and the wall reinforcement comprised at least three sizes of mild steel reinforcement no larger than no. 3 (10M) bars.

The structure may have had sufficient area of walls to sustain the lateral forces from the earthquake, but the connections were insufficient in strength and ductility to support them against either in-plane or out-of-plane forces. The panel design deficiencies in thickness and in reinforcement may also have contributed to out-of-plane failures.

Moment-resisting frames

There were examples of precast concrete column and beam framing that had cast-in-place concrete closure joints that created continuity and formed moment-resisting frames. These systems generally performed well.

One industrial building that used long-span girders with wet-cast joints was formed with 52 ft × 79 ft (16 m × 24 m) bays. The framing included precast concrete girders in both the longitudinal and transverse directions, bearing on top of precast concrete columns with wet-cast joints. **Figure 17** shows one interior joint. The roof sloped from the sides to the center girder line. The roof deck was supported between the girders with simply supported precast concrete joists. The roof plane, in lieu of a diaphragm, had diagonal bracing below the roof joists that was connected between girders to plates and gussets. The diagonal bracing was also composed of precast concrete joists, similar to those in **Fig. 18**. The only significant damage to this structure from the earthquake was that all of these diagonal braces disconnected at the plates at the girders and fell.

The girders were formed as I-shaped sections with rectangular end blocks, similar to bridge girders used in the

Figure 17. Wet-cast joint between girders and column in an industrial building that performed well during the earthquake.

Figure 18. Diagonal precast concrete bracing for precast concrete girder roof framing in an industrial building that did not experience structural damage during the earthquake.

Figure 19. Total–precast concrete school framing system used in more than 40 schools that experienced no damage from the earthquake.

Figure 20. Interior of Espacio Riesco Exhibition Hall with cantilevered column framing.

United States. This building was clad using vertically spanning precast concrete double-tee wall panels. The panels were connected to the structure with long threaded rods that projected across the open void formed by the I shape and bolted through the webs. These connections provided out-of-plane support, but they did not engage the cladding as shear walls to provide any assistance to the LFRS.

The field-cast joints for the building were described as having reinforcement projecting from the ends of the girders into the space over the tops of the columns with bar laps and hooks that were engaged by the closure pour.

Other examples with similar framing and bracing were found at the Weir Vulco plant. Figure 18 shows the precast concrete diagonal framing in place. These buildings did not experience damage to the structural systems. Some bracing for these buildings was provided by shear walls that framed large door openings with drop-in walls over the doors. Precast concrete column and beam framing made continuous at the columns was also used for the construction of 40 total–precast concrete schools and buildings at five universities. No failures were reported in any of these buildings.

The diaphragms and continuity of joints in these systems were developed with cast-in-place concrete topping for floors and wet joints. The floor framing was constructed with precast concrete double-tees and tapered end stems and flanges that formed the pocket for the wet joint. **Figure 19** shows a view of the underside of this framing.

Cantilevered column systems

There were several examples of industrial buildings with precast concrete framing supported by cantilevered columns as the LFRS.

One example in Santiago was a large exhibition hall. The structure was framed with three consecutive bays, each 130 ft (40 m) long. It had eight 40-ft-wide (12 m) bays spanned by spread precast concrete beams with a trapezoidal section. **Figure 20** shows an interior view of the framing. The 130-ft girders are tapered I-beams that form gable roofs. These beams bear on 40-ft columns and are held with vertical rods that pin the ends to the bearings. The columns are 35 in. (900 mm) square. Without moment continuity between the ends of the beams or between the beams and the columns, the lateral support for the structure is provided by only the cantilever action of the columns at the footings. The footings are not tied together with grade beams, but the columns were designed for a combined lateral force equal to 25% of the weight.

The spread precast concrete beams do not form moment-resisting frames, but they have wet-cast connections at

Figure 21. Interior view of the roof framing at a can factory.

Figure 22. Base isolation bearings at Weir Vulco.

their bearings on the roof girders, so the secondary framing is made continuous. This detail adds some redundancy to the roof system and contributes to the overall structural integrity. The spread-beam system, however, does not form a continuous diaphragm capable of redistribution of the forces. The building suffered no structural damage to the primary LFRS.

This building was clad to about two-thirds of the exterior wall height with horizontal precast concrete walls. These walls were not intended to act as shear walls, and some of the upper panels fell from the structure as the cladding connections failed. This aspect is discussed in the section on precast concrete cladding.

Another example of precast concrete framing with cantilevered columns was found in Coronel, south of Concepción, at the Parque Industrial Escuadrón, adjacent to the failed gable frames described earlier. This recent construction also used long-span gable-shaped roof girders and spread-beam framing to form a warehouse for the fish meal operation. The gable-shaped girders are pierced with round holes in the webs to reduce their weight. Again, the trapezoidal spread roof beams were made continuous across these girders with cast-in-place concrete joints. One section of this building was about 52 ft (16 m) tall, with beams framing with pinned end connections at two levels above and below the girders. This building survived the earthquake without damage.

A second example in Coronel was found at a manufacturing facility. The framing was similar to that of the fish meal facility. **Figure 21** shows an interior view of this building showing the gable-shaped girders and spread trapezoidal roof beams. Field-cast joints created continuity in the roof beams across the girders.

This structure was damaged in three areas. In one location there was a long exterior cantilevered canopy over a loading area that was framed with steel beams. The beams were fixed to the precast concrete columns with exterior plates bolted around the columns. The earthquake caused the canopy to sag and the columns to crack near the beam connections. The cantilevered behavior of the columns resulted in flexural cracks near the bottoms of the columns. The team also found that there were some local spalls at the bearing of a roof beam where the width of the beam spanned across the joint between the ends of the roof girders. The observed damage in these areas was not severe and was being repaired.

The design of cantilevered column systems was shown to be effective.

Advanced seismic-force-resisting systems

There were several examples of framing systems using advanced concepts that proved effective during the earthquake.

Base-isolated offices As a demonstration of laminated base-isolation rubber bearings, the manufacturer constructed a total–precast concrete office building supported on slide bearings at the corners and on base isolators at the interior bays. **Figure 22** shows a view of the isolation bearings on one side.

The building is two bays by five bays, with a square module of 26 ft (8 m). The structure is only two stories tall, but the company has supplied isolation bearings to fifteen other buildings with similar design. The structure experienced no damage during the earthquake, though the slide bearings showed movement of about 5 in. (130 mm) in both longitudinal and transverse directions. The owner reported that books standing on end in the structure did not fall over.

It was reported that these bearings were used in some residential buildings, some bridges, and at buildings at the

Figure 23. Interior view of a convention hall with tilted braced frames.

Figure 24. Exterior view of a convention hall with braced frame and cable-stay supports.

Catholic University and the University of Chile. They were used in a dock at the port of Coronel, which was reported to be the only dock not damaged by the earthquake.

Unbonded prestressed concrete frames and walls A precast concrete manufacturer constructed a five-story structure at its convention/exhibition site that uses unbonded post-tensioned walls and frames following the research of the PCI PRESSS (Precast Seismic Structural Systems) program. The structure is braced in the short direction by post-tensioned shear walls placed at the ends of the building. The post-tensioning strands are located near the center of the walls. In the other direction, there are three bays framed with unbonded post-tensioned moment-resisting frames. Although the erection of the structure was complete, the building was unfinished. The first floor was in operation as a kitchen for the convention center. The upper floors were not yet completed. The structure experienced no damage from the earthquake.

Reinforced concrete braced frames with cable-stayed roof The main building in the convention center complex is a large conference hall constructed with precast concrete braced frames. The frames are tilted in from the side walls so that the clear span at the floor level is 200 ft (61 m) but the girder span for the roof is reduced to 160 ft (49 m). **Figure 23** shows an interior view of this framing.

The clear height under the roof girders is 40 ft (12 m), and they are 5 ft (1.5 m) deep. Roof beams span between the girders to support the roof decking. Because of the long spans of the girders, there is additional support provided by cable stays. To hold these stays above the roof, columns were added above the top intersection of the tilted braced frame columns; these added columns lean outward. The joints tying the lower and upper columns together were made with field-cast concrete. **Figure 24** shows an exterior view of the cable stays, braced frames, and columns.

At one end of the hall, bracing columns extend to the edge girder. At the other end, a large room for staging and service support is framed with seven sides and clad to about half the wall height with horizontal stacked precast concrete wall panels. These cladding panels were not used as lateral bracing for the structure and were connected to columns with erection angles between slotted inserts. The slots are oriented horizontally in the walls and vertically in the columns, apparently to allow compensation during erection for casting tolerance between walls and interior framing. The roof for this side room is supported with tapered precast concrete girders that span the width of the room and are supported by, and cantilever over, another interior long-span girder. This girder spans 160 ft (49 m) and is 8 ft 2 in. (2.5 m) deep.

The only damage to this building from the earthquake was from failure of the cladding panel connections. It was reported that the top panels pulled out of plane and fell off the structure. Some of the panels shown in the photograph were replaced temporarily while waiting for new panels with additional connections to be fabricated.

Precast concrete stadiums

The PCI team investigated stadiums framed with precast concrete columns, beams, rakers, and risers in Chillán and in nearby Chillán Viejo.

The stadium in Chillán was framed with precast concrete for the seating areas on four sides enclosing the playing field. The seating was shaded with a fabric roof within steel frames supported by cantilevered steel columns attached to the tops of precast concrete columns on the perimeter; these were braced by the raker beams and transverse beam framing. The project was planned on a tight schedule, and when the precast concrete manufacturer could not supply sufficient components, the framing of the press box and supporting building was converted to cast-in-place concrete, with a vertical line of separation at the back of the seating area.

The precast concrete framing included rakers, columns, beams, and single-step risers. **Figure 25** shows a view of the framing during erection. The precast concrete framing was tied transverse to the rakers by beams with welded connections. With the exception of the failed roof structure falling on the seating, the precast concrete framing withstood the earthquake with only minor damage to bearing surfaces, which showed some cracking. The primary failure occurred at the connections at the tops of columns where the steel framing for the fabric roof tore from column-base connections and at some of the bracing cable anchor connections. The failures occurred only at cast-in-place concrete columns on the press box side of the stadium.

The stadium in Chillán Viejo was a smaller structure with precast concrete columns, rakers, and risers on opposite sides of the playing field. The framing was a simple single-span raker on exterior columns. On one side, the seating was backed up with a press box structure.

Damage to this stadium appeared to be relatively minor. There was a spall at the bearing of a riser stem at the top of a raker where the bearing area was not confined, but the concrete appeared to be intact over most of the bearing length. There were also cracks at the bearing of a raker beam at a column corbel, but most of that bearing appeared intact.

It appeared that the precast concrete framing performed well during the earthquake.

Performance of precast concrete cladding and cladding connections

In Chile, precast concrete cladding panels are used on industrial buildings and on some low-rise office buildings. Precast concrete cladding was not observed on high-rise structures. Most of the panels observed performed well. There were some cases where connections between the cladding and the supporting structure failed.

Most of the precast concrete cladding panels observed were nonstructural. These panels were subject only to inertial seismic forces and wind loads. For effective support, the connections of these panels should accommodate movement of the supporting structure. Without this flexibility, cladding panels can attract unintended forces. Examples of successful and unsuccessful performance were observed.

There were several warehouse-type buildings using vertical double-tee wall panel cladding that performed well. These

Figure 25. Stadium with precast concrete rakers on precast concrete columns.

buildings had lightweight non-diaphragm roofs and the lateral forces were resisted by the column-beam framing system, sometimes with roof-level diagonal bracing, and a few cast-in-place concrete shear walls. The double-tees are commonly about 8 ft (2.4 m) wide and 30 ft (9.1 m) tall, with conventional (nonprestressed) reinforcing.

The team investigated two buildings with double-tee cladding. One used flat precast concrete panel cladding at the corners and double-tee walls along the sides. The other used double-tee wall panel cladding that included walls supported above a wide loading dock opening. In these two examples, the full-height double-tee wall panels were supported on the foundation and with projecting reinforcing cast into the floor slab. Both the floor slab and an exterior slab were cast against the base of the wall panel. Near the top of each double-tee leg, there were long bolt tiebacks that projected through the interior perimeter beams (**Fig. 26**). The bolts were more than 1 ft (0.3 m) long to provide out-of-plane restraint while allowing movement parallel to the wall of the framing system and the roof system without transmitting force to the wall system.

At the convention center complex described previously, the exhibition and convention halls were clad with long horizontal precast concrete panels. The panels were stacked two to four units high, with the primary weight transferred through the panel below and then to the foundation. One of the buildings had sloped precast concrete columns, creating a braced frame, so that part of the panel weight was carried by its connections to the columns (Fig. 23 and 24). The end annex to that building (**Fig. 27**) and the adjacent exhibit hall had vertically stacked panels. In neither case did the precast concrete walls reach the level of the roof. Metal cladding was used to complete the enclosure.

The wall panels were attached to the concrete columns with long slotted embedments (**Fig. 28**). The embedments are commercial inserts commonly used for precast concrete connections. The slotted inserts are oriented vertically

Figure 26. Double-tee panel connection to perimeter beams used in industrial buildings.

Figure 27. Annex of Preansa Convention Hall uses horizontal precast concrete panels.

Figure 28. Slotted inserts to connect wall panels to concrete columns and the deformation of the slotted inserts due to the earthquake load.

in the columns and horizontally in the wall panels to provide ample alignment tolerance. The connection appeared to be for out-of-plane forces only. Although the photos show the precast concrete wall panels in place, some of those panels fell away from the buildings during the earthquake and were replaced. As designed and installed, these connections did not have sufficient strength to withstand the earthquake forces. The slotted insert embedment had deformed at the lips and allowed the bolts to pull out. One large all–precast concrete multioccupant structure that was virtually destroyed was described earlier in this report. The building's nonstructural precast concrete cladding was damaged or simply collapsed. Cladding on the building was damaged as the supporting structure failed. Exterior panels fell away due to connection failures at the roof level.

Precast concrete cladding was used on the steel-framed structure of a building supply warehouse store in Concepción. Panel collapse at this structure appeared to be caused by the failure of the supporting structure. It was not possible to determine whether inertial forces from the cladding contributed to the failure.

An office structure in Concepción clad with precast concrete panels appeared intact, though much of the infill glass was broken. Precast concrete cladding was also used on the base-isolated Weir Vulco building. The base isolation of that building protected the cladding as well as the structural precast concrete framing. Other buildings with precast concrete wall cladding were observed from a distance to have been damaged, but with access limited, the configuration and the extent of damage could not be determined.

In general, the precast concrete cladding panels in Chile performed well when the effects of and requirements for seismic resistance were considered in design and detailing.

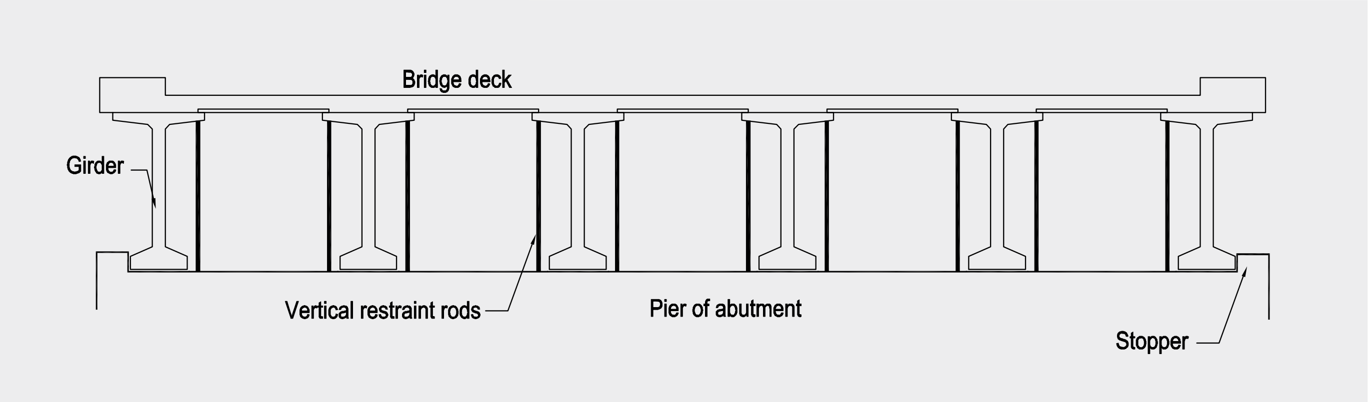

Figure 29. Bridge section showing recent construction practice that eliminates the transverse diaphragm and uses vertical steel rods to prevent overturning.

Precast concrete bridges

Many highway bridges in Chile are constructed with precast concrete I-girders or bulb-tee sections. Many of these crossings performed well, but there were also many notable failures that shared common characteristics.

Precast concrete bridge construction

Moderate-span precast concrete bridges in Chile are constructed using I-girder or bulb-tee girder shapes, similar to AASHTO sections used in the United States. The design includes cast-in-place concrete diaphragms between girders. The girders bear in pockets with direct lateral restraints against displacement at each girder. The diaphragms tie the girders together laterally and prevent overturning from lateral loads above the bearings.

Local engineers reported that design practice for these bridges changed in the late 1990s following Spanish practice so that diaphragms providing lateral support between girders were reduced or eliminated. Lateral support at the bearings was reduced to end stoppers at the ends of the piers or abutments, constructed with small reinforced concrete sections projecting above the beam bearing surface. Vertical steel rods from the bearing to the underside of the upper flange of the girders compensate for the loss of overturning resistance. **Figure 29** shows a section representing these features of precast concrete bridge design.

The bridges with designs based on the more recent practice experienced more damage than bridges constructed with traditional details. The more recent bridges suffered lateral displacements at the bearings, failure in end stoppers, and some overturning of the beams.

Bridges with skewed bearings failed because a lack of lateral restraint permitted the global rotation of the bridge and allowed the beams to fall from the bearings. The absence of lateral restraints at the bearings is not sufficient to provide a mechanism for collapse. Although bridges with this configuration are more susceptible to loss of bearing, the geometry of the rigid concrete deck must contact the piers or abutments and cause some lateral displacement of the support for the rotation to continue. This strongly suggests that a lack of longitudinal restraint of these bridge girders at the abutments or piers at one end of the span at least contributed to these failures.

Figure 30. Failed bearing in the Llacolen Bridge.

Bridge inspection

The PCI team inspected two bridges in Concepción. The first bridge was the Llacolen Bridge, which includes several moderate-length spans across the Bio Bio River. This bridge was constructed without diaphragms between the girders at the piers. The construction included the vertical restraint bars enclosed in galvanized steel tubes. **Figure 30** shows the bearing surface with concrete debris and twisted reinforcing from the failed girders and deck. The photo also shows the bent and twisted galvanized tubes that held the failed restraint bars.

As seen in the photo, the bridge girders pulled away from the bearing and dropped the end of the span. This was not a skewed span, but the span lacked both lateral and longitudinal restraint at this bearing. The length of the

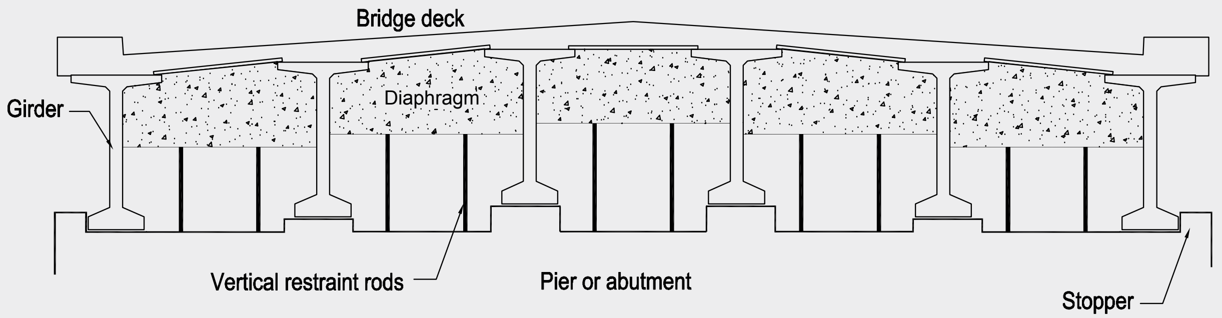

Figure 31. Bridge section at the Bio Bio River crossing with partial diaphragms, vertical restraint bars, and end stoppers.

bearing was not sufficient for the longitudinal displacement between the piers. This was one of several failed multispan bridges that dropped the ends of spans.

Figure 30 also shows an elevated bearing surface that is a reminder that actual construction geometry is often not as simple as the illustration of the geometry shown in the earlier figures.

Figure 31 shows another bridge section that includes the center crown and cross slope for drainage that must often be included. It is common for the beam bearings to be stepped to provide the crowned shape or to provide for cross slope or superelevation in the roadway above. These features of practical geometry can result in the loss of lateral restraint to the girders; this was observed in the Llacolen Bridge. The second bridge was a multispan bridge crossing an inlet of the Bio Bio River.

This bridge had some of the characteristics of more recent bridge construction details but did include partial-depth diaphragms between the girders at the bearings. Figure 31 also includes the diaphragms, the vertical restraint bars, and the end stoppers. **Figure 32** shows a view of the bearing at two interior girders.

Figure 32. Lateral displacement due to earthquake forces of the Bio Bio River bridge with partial diaphragms and vertical restraint bars.

The abutment bearing was cast as a level ledge, and then the bearing blocks with varying thicknesses were placed to form the final bearing elevations. This convenient method used to construct the correct bearing elevations lacks the lateral restraint of bearing details using pockets. The earthquake forces caused lateral displacement and the failure of the stopper at one end of the pier. The failure did not indicate a lack of reinforcing, though the horizontal confinement did not appear to contain all of the vertical reinforcement. The design, however, imposed all the requirement of lateral restraint on the end stopper. **Figure 33** shows the failed stopper at the abutment. This view also shows a wide crack in the bottom of the edge girder from the impact with the stopper.

A bridge in Santiago designed with vertical restraint bars attached to precast concrete girders and bridge pier showed no evidence of displacement. Damage at this bridge was seen in subsidence of the fill that formed the bridge approach and embankment at the grade separation.

Building code

Buildings and other structures in Chile must be designed and constructed in compliance with the Chilean Building Code NCh433-2009.[6] This code is applied in addition to the specific design code for each of the materials and aims to achieve structures that meet the following objectives:

- resist moderate intensity of seismic actions without damage
- limit damage to nonstructural elements during earthquakes of moderate intensity
- prevent collapse during earthquakes of severe intensity, even though they show some damage

Compliance with the provisions of this code does not guarantee that the objectives will be achieved.

In particular, the provisions for reinforced-concrete-wall buildings are based on their satisfactory behavior dur-

Figure 33. Failed stopper at the abutment of the Bio Bio River bridge, with a crack in the bottom of the girder.

Table 2. Soil parameters for base shear calculation in NCh433-2009

Soil type	S	T', sec	n
I	0.9	0.20	1.00
II	1.0	0.35	1.33
III	1.2	0.85	1.80
IV	1.3	1.35	1.80

Note: n = soil parameter for base shear calculation indicated in NCh433-2009; S = site coefficient; T' = soil parameter for base shear calculation indicated in NCh433-2009.

Table 3. Maximum values of seismic coefficient C in NCh433-2009

R	C_{max}
2	$0.90SA_0/g$
3	$0.60SA_0/g$
4	$0.55SA_0/g$
5.5	$0.40SA_0/g$
6	$0.35SA_0/g$
7	$0.35SA_0/g$

Note: A_0 = maximum effective acceleration value; C_{max} = maximum seismic coefficient; g = acceleration due to Earth's gravity; R = reduction factor ranging from 2 for structural systems of limited ductility to 7 for ductile structural systems; S = site coefficient.

Where is the referencef for these tables?

ing the earthquake of March 1985. Those buildings were designed in accordance with NCh433-1972.[18]

The Chilean seismic code NCh433-1996[11] was in effect until it was replaced by the 2009 update (NCh433-2009)[6] shortly before the earthquake. A 2010 update has been developed in direct response to the earthquake.

NCh433-2009[6] has four building categories: A, B, C, and D. These are comparable to ASCE 7/05 occupancy categories IV (essential facilities, hazardous facilities), III (high-occupancy buildings where many people congregate in one space at one time), II (standard-occupancy buildings such as office buildings and apartments), and I (miscellaneous-occupancy buildings where no life safety is at stake), respectively. The importance factor I for building types A, B, C, and D is 1.2, 1.2, 1.0, and 0.6, respectively.

The country is divided into three seismic zones: zone 1 is along the foothills of the Andes, zone 3 is along the Pacific coast, and zone 2 is between zones 1 and 3. The maximum effective acceleration values A_0 corresponding to zones 1, 2, and 3 are $0.20g$, $0.30g$, and $0.40g$, respectively.

NCh433-1996 considers four soil types: I, II, III, and IV (**Table 2**). These are comparable to soil profile types S_1, S_2, S_3, and S_4, respectively, in UBC editions before 1997. Soil type I is rock; soil type IV is soft soil.

The base shear Q_o is determined from Eq. (1).

$$Q_o = CIP \tag{1}$$

The seismic coefficient C is obtained from Eq. (2).

$$C = \frac{2.75A_0}{gR}\frac{\left(T'\right)^n}{\left(T^*\right)^n} \tag{2}$$

In no case shall the value of C be less than $A_0/6g$.

The value of C need not be greater than that indicated in **Table 3**.

There are no specific provisions for or prohibition of buildings with structural irregularities.

ACI 318-95 seismic provisions were referenced by NCh433-1996[11] in annex B, except the confinement requirements for wall boundaries were specifically exempted in clause B.2.2 as previously discussed. This clause has

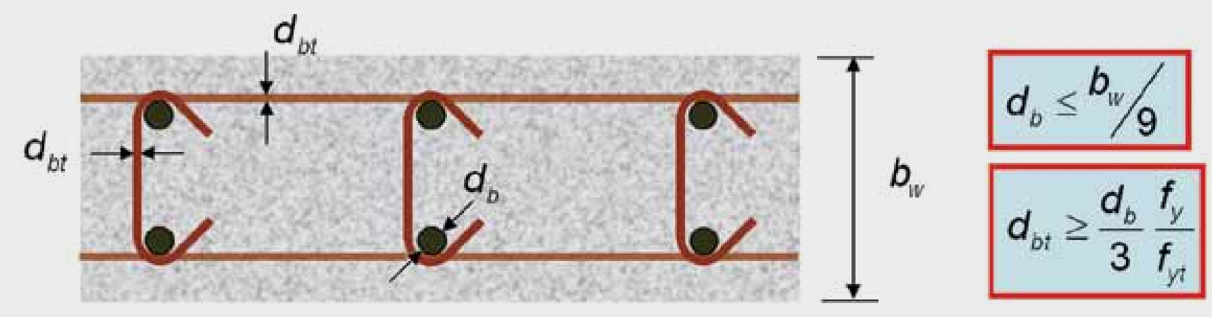

Figure 34. Longitudinal and transverse reinforcing bar diameter limitations. Note: b_w = thickness of wall; d_b = nominal diameter of longitudinal bar; d_{bt} = nominal diameter of transverse bar; f_y = yield strength of longitudinal reinforcement; f_{yt} = yield strength of transverse reinforcement.

been deleted from NCh433-2009.[6]

The following changes[19] to NCh433-2009[6] were being considered and were in draft form at the time of the team's visit:

- Limit axial forces on columns and walls that are subject to lateral displacements to $0.35f_c'A_g$.
- All hooks on hoops and cross ties of confinement reinforcement should be 135° rather than 90°.
- Confine at least $0.15\ell_w$ from each edge and laterally support every vertical bar, not just every other one.
- Apply capacity design concepts to determine axial force and shear, considering the effect of connection with the slabs.
- Add slenderness restrictions to avoid transverse bending of boundary elements and the panel. For this, study the New Zealand[19] and Canadian[20] code recommendations.
- Revise the displacement spectrum.
- Avoid adding to the resistance side; add to the demand side. Study the effect of resistance (strength) on displacement demand.
- Optional displacement-based design has been introduced in annex B.

Patricio Bonelli, a professor at Universidad Técnica Federico Santa María, also shared the following recommendations and reflections:

- Carefully study the demands of displacement and rotation. In Concepción, 15 cycles of 0.2*g* acceleration were observed at a period of 1.5 sec.
- Microzonation of cities is desirable.
- Studies should be nonlinear-analytical and experimental. Analytical results should be compared with observations and new findings that confirm or reject the proposed explanations.
- A building responds to an earthquake with the structure as constructed and with material properties that exist at the time of the earthquake. This may be obvious but is often ignored.

At a presentation before ACI 318 subcommittee H in October 2010, Bonelli discussed the following changes made to ACI 318-08[16] requirements in the emergency changes to NCh433-2009[6,21] and NCh430-2008:[22,23]

1. The whole flange width of a flanged section (T, L, C, or other cross-sectional shapes) must be considered in calculating combined flexural and axial load strength.
2. The contribution of the total amount of longitudinal reinforcement must be considered in determining combined flexural and axial load strength.
3. Longitudinal reinforcing bar diameter must be less than or equal to one-ninth of the least dimension of the boundary element (wall thickness) (**Fig. 34**).
4. Transverse reinforcing bar diameter must be greater than or equal to one-third of the diameter of the restrained longitudinal bar (Fig. 34).
5. Transverse reinforcement must be anchored to extreme longitudinal bars in a wall.
6. Standard hooks must be used with transverse reinforcement as defined in section 7.1 of ACI 318-08: 135° or 180° bend plus $6d_b$ extension, but not less than 3 in. (75 mm) at the free end of the bar. In ACI 318-08,[16] a standard hook is defined as a 180° bend plus $4d_b$ extension or a 90° bend plus $12d_b$ extension. Thus, these two requirements are contradictory. The second requirement (135° or 180° bend plus $6d_b$ extension),

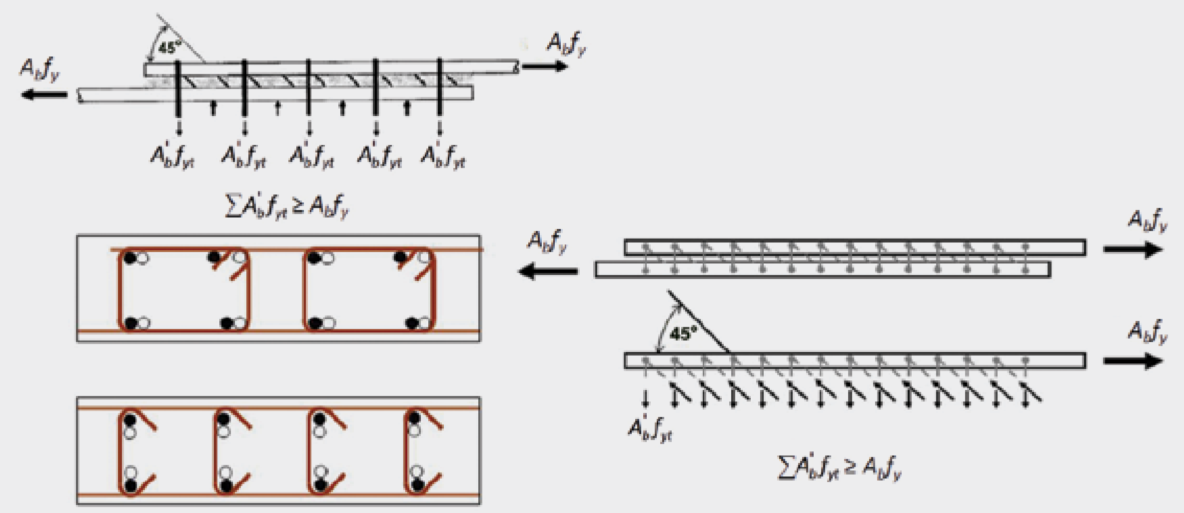

Figure 35. Transverse reinforcement through the length of lap splice. Note: A_b = area of an individual longitudinal bar; A'_b = area of an individual transverse bar; f_y = yield strength of longitudinal reinforcement; f_{yt} = yield strength of transverse reinforcement.

being more restrictive than the first, is assumed to govern.

7. In special structural walls, the net tensile strain in the extreme tension steel ε_t must be equal to or greater than 0.004 when the concrete in compression reaches its assumed strain limit of 0.003. (Section 10.3.5 of ACI 318-08 requires ε_t at nominal strength to be not less than 0.004 for nonprestressed flexural members and nonprestressed members with factored axial compressive load less than $0.10 f'_c A_g$.)

8. The transverse dimension (thickness) of special structural walls must be greater than or equal to one-sixteenth of the lateral unsupported member length under compression, $\ell_u/16$.

9. In special structural walls, when shear has not been calculated using capacity design rules, the maximum shear obtained from design load combinations that include load effects of earthquake *E* shall be calculated with *E* assumed to be 1.4 times that prescribed by the legally adopted general building code for earthquake-resistant design. ACI 318-08 section 21.3.3(b) lists the comparable factor as 2, which is applicable to beams and columns of intermediate moment frames.

10. Transverse reinforcement through the length of lap splices of longitudinal bars in walls must satisfy items 4, 6, and 11 of this list when either (a) or (b) is true.

    a) The longitudinal reinforcement ratio, defined as $\Sigma A_b/hs$, is greater than $2.8/f_y$.

    b) The cover of a longitudinal bar with nominal diameter d_b is less than $2d_b$.

11. Transverse reinforcement through the length of lap splices of longitudinal bars in walls must satisfy the condition in **Fig. 35**.

12. In section 21.9.6.2 of ACI 318-08[16] on boundary elements of special structural walls, the lower-bound limitation of 0.007 on δ_u/h_w in Eq. (21-8) shall not apply.

13. Section 21.9.6.4(a) of ACI 318-08[16] may be replaced by the following:

    The boundary element shall extend horizontally from the extreme compression fiber a distance not less than c_c determined from Eq. (3).

$$\frac{c_c}{\ell_w} = \frac{c}{\ell_w} - \frac{1}{600\dfrac{\delta'_u}{h'_w}} \tag{3}$$

    When this option is applied, the term δ_u/h_w in ACI 318-08[16] section 21.9.6.2 must be replaced by δ'_u/h'_w. The lateral design displacement δ_u comes from NCh433-1996[11] (modified in 2009[6]) section 5.9.5.

14. Replace ACI 318-08[16] section 21.9.6.2(b) with "The boundary element reinforcement shall extend vertically from the critical section a distance not less than ℓ_u."

15. Transverse reinforcement in boundary elements in walls, when required, must satisfy ACI 318-08 section 21.9.6.4 and (a) and (b).

a) The spacing of cross ties or legs of rectilinear hoops h_x within a boundary element in a wall shall not exceed the smaller of 8 in. (200 mm) and the least dimension of the boundary element.

b) The spacing of transverse reinforcement in a boundary element in a wall shall not exceed the smaller of six times the diameter of the smallest longitudinal bar and half the minimum boundary element dimension.

16. Replace ACI 318-08[16] section 21.9.6.5(a) with the following:

    a) Where the longitudinal reinforcement ratio at the wall boundary is greater than $2.8/f_y$, boundary transverse reinforcement shall satisfy sections 21.6.4.2 and 21.9.6.4(a) or item 13 of this list. The maximum longitudinal spacing of transverse reinforcement in the boundary shall not exceed 8 in. (200 mm).

    b) Where vertical reinforcement can yield, the maximum longitudinal spacing of transverse reinforcement in the boundary shall not exceed the smaller of six times the longitudinal bar diameter and 8 in. (200 mm).

Conclusion

The majority of structures performed acceptably or better, considering the severity of the 2010 Chile earthquake. The number of deaths and the amount of property loss, while quite significant, were not disproportionate to the severity of the earthquake. Much of this is attributable to Chile's history of adoption and implementation of adequate building codes.

The PCI team concentrated on precast concrete structures. With the exception of the out-of-date gable frame system observed in one location south of Concepción, the precast concrete building systems generally performed well. In some cases, the LFRS performed, but the absence or weakness of diaphragm framing resulted in local failures. Where lateral forces were resisted by cantilevered columns and distribution of loads through diaphragm action was not essential, the structural framing of the buildings did not experience significant damage.

Some of the buildings inspected showed the success of advanced precast concrete seismic systems, which reflects research conducted in the United States. Some used technology associated with other materials on the U.S. market. The example of the reinforced concrete braced frames showed the success of a system that is not included in the defined systems in ASCE 7-10.[8] The PCI team found a mature and sophisticated precast concrete industry that has successfully considered and solved problems of earthquake resistance without some of the constraints imposed on U.S. practice by restrictive building code provisions.

In general, precast concrete cladding panels performed well in Chile as long as the effects of and requirements for seismic resistance were considered in design and detailing.

The bridge infrastructure in Chile experienced large-magnitude shaking, often larger than what the bridges were designed for, with varying degrees of damage. With the exception of the more recently constructed bridges in which diaphragms were reduced or eliminated, the precast concrete bridges observed by the PCI team generally performed well. In some cases, the girders performed well but the absence or weakness of diaphragms or lateral restraint resulted in failures at bearings or piers. Failures at bearings and piers and the rotation of spans causing loss of bearing were not confined to bridges constructed with precast concrete girders but were also seen in steel-girder bridges constructed with similar end details.

The 2010 emergency changes to Chile's building code have far-reaching implications for the special structural wall design provisions in ACI 318-08[16] section 21.9. Changes will be considered for possible inclusion in the next version of ACI 318 and, if adopted, may have significant effects on design practice in the United States.

Acknowledgments

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The PCI team consisted of Ned Cleland of Blue Ridge Design in Winchester, Va.; Susan Dowty of S. K. Ghosh Associates Inc. in Aliso Viejo, Calif.; S. K. Ghosh of S. K. Ghosh Associates Inc. in Palatine, Ill. (team leader); Ray McCann a structural engineer in Napa, Calif.; and Dante Sanguineti of Pomeroy Corp, in Perris, Calif. Augusto Holmberg of Instituto del Cemento y del Hormigón de Chile was the PCI team's local contact and host. Patricio Bonelli, a professor at Universidad Técnica Federico Santa María, provided much valuable information concerning Chile's building code and changes to that document. He also led three of the team members on a tour of Viña del Mar. His help is gratefully acknowledged. The contribution of the section on cladding by Ray McCann and review of the original version of this report by all team members are much appreciated. The authors are grateful to Prabuddha Dasgupta of S. K. Ghosh Associates Inc., whose help in the preparation of the manuscript was invaluable.

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Notation

A_0 = maximum effective acceleration

A_b = area of an individual longitudinal bar

A_b' = area of an individual transverse bar

A_g = gross area of concrete section

b_w = thickness of wall

c = largest neutral axis depth calculated for the factored axial force and nominal moment strength consistent with δ_u

c_c = extension of the confined area measured from the extreme compression fiber

C = seismic coefficient

C_{max} = maximum value of seismic coefficient

d_b = nominal diameter of longitudinal bar

d_{bt} = nominal diameter of transverse bar

E = effects of earthquake

f_c' = compressive strength of concrete

f_y = yield strength of longitudinal reinforcement

f_{yt} = yield strength of transverse reinforcement

g = acceleration due to Earth's gravity

h_w = height of entire wall from base to top or height of the segment of wall considered

h_w' = height of wall between top and considered level

h_x = spacing of cross ties or legs of rectilinear hoops

I = importance factor for building categories, as specified in NCh433

ℓ_u = lateral unsupported member length under compression

ℓ_w = length of wall

M = moment magnitude of earthquake

n = soil parameter for base shear calculation indicated in NCh433-2009 (Table 3)

P = total weight of the building above the base level

Q_o = base shear

R = reduction factor ranging between 2 for structural systems of limited ductility and 7 for ductile structural systems in NCh433

s = center-to-center spacing of longitudinal reinforcement

S = site coefficient

S_a = spectral acceleration

S_d = spectral displacement

T = time

T' = soil parameter for base shear calculation indicated in NCh433-2009

T^* = period of mode with the highest translational equivalent mass in the direction of analysis

β = damping coefficient

δ_u = lateral design displacement

δ_u' = design drift measured between the top and the considered level

ε_t = net tensile strain in the extreme tension steel

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Abstract

This paper reports on the findings and observations of the team sent by PCI to investigate damage caused by the February 2010 earthquake in Chile. The paper concentrates on the performance of precast concrete structures, although some other aspects are also included. The majority of structures performed acceptably or better, considering the severity of the earthquake. Much of this is attributable to Chile's history of adoption and implementation of adequate building codes. The success of advanced precast concrete structural systems, based on research in the United States, was demonstrated. The Chilean precast concrete industry is mature, sophisticated, and, in the absence of constraints imposed by restrictive building codes, quite innovative. Code implications of lessons from the February 2010 earthquake are also discussed.

Keywords

Bridge, cladding panels, code, construction, earthquake, lateral resistance, seismic, structural system, structural wall.

Reader comments

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