# Nonductile Concrete Frames

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Figure 1. Kaiser Permanente. Source: NISEE-PEER, University of California, Berkeley.

The Northridge earthquake struck the greater Los Angeles area during the early morning hours of January 17, 1994. The earthquake was responsible for approximately 60 deaths, more than 9,000 injuries, and an estimated $20 billion in damages. Significant ground shaking occurred over a wide area and exceeded design code values in many locations. Numerically, most of the damage was to wood-frame residences, but upwards of 200 concrete buildings were red-tagged. The Northridge earthquake was the first big test of pre-1980 concrete buildings and post-1980 buildings designed using updated code provisions following the 1971 San Fernando earthquake. The 1971 San Fernando earthquake exposed the deficiencies of the building codes in place at the time, particularly related to concrete. The collapses of the Olive View Medical Center and the Veterans Administration Hospital that occurred as a result of the San Fernando earthquake are famous examples of the hazards posed by "nonductile" concrete (NDC) buildings. Several of these NDC buildings collapsed or were severely damaged during the Northridge earthquake as well, including the Kaiser Permanente Office Building (*Figure 1*) and Saint John's Hospital. This article discusses building code provisions for concrete structures, the performance of non-ductile concrete frame structures in the Northridge earthquake, associated changes made to the building code after, and retrofit ordinances being considered today for existing non-ductile concrete buildings.

## Seismic Code Background

Seismic building codes are continually evolving based on new information gained through research and observation of building performance during earthquakes. The great 1906 earthquake prompted the City of San Francisco to include earthquake design load requirements for buildings. In July 1959, the SEAOC Seismology Committee published the first edition of the *Blue Book*, officially titled *Recommended Lateral Force Requirements*. This "code" was the first to formalize the relationship between earthquake demands, building period, and the ductility of the lateral-force-resisting system. Preference was given to moment-resisting space frames for lateral resistance relative to bearing walls through the use of a lower "K" factor, which can be thought of as proportional to the inverse of the "R" factor in ASCE 7.

Meanwhile, researchers and practitioners were beginning to understand the advantages of ductile behavior and began testing and quantifying ways to provide ductility in concrete structures. In 1961, Blume, Newmark, and Corning published *Reinforced Concrete Buildings for Earthquake Motions*. The book provided design methods and detailing principles for ensuring ductile behavior such as maximum allowable steel percentages, providing closely-spaced closed ties in columns and beams, and providing continuous top and bottom steel for stress reversals. Unfortunately, the concept of a ductile moment-resisting frame did not find its way into codes until the 1967 *Uniform Building Code* (UBC). However, ductile frames were required only for buildings greater than 160 feet in height. These provisions required smaller tie and stirrup spacing along the lengths of moment frame columns and beams, respectively, and special transverse joint reinforcement.

The San Fernando earthquake provided the impetus to update code requirements to ensure the ductile behavior of concrete structures, and the 1976 UBC is considered to be the first code to provide seismic resistance of concrete buildings similar to current code. Given the lag in construction year relative to design year, the benchmark that most engineers use for determining if a building is likely NDC is 1980.

The most common types of deficiencies in NDC buildings include:

1) Beam and column stirrups and ties spaced relatively far apart, causing shear failures and lack of core confinement.
2) Use of 90-degree bends on closed stirrups instead of 135-degree hooks.
3) Lack of joint shear reinforcement.
4) Inadequate lap splices and locating them in regions of high flexural stress.
5) Weak columns – strong beams.
6) Slab-column punching shear.
7) Plan or vertical irregularities resulting in torsion or soft or weak stories.

## Damage to Concrete Frame Buildings

The damage caused to pre-1980 concrete buildings by the Northridge earthquake was significant but not a surprise. In general, pre-1980 shear wall buildings met life safety and collapse prevention performance objectives.

The Sherman Oaks Towers was a 12-story building designed to the 1964 Los Angeles City Code. The structure comprised flat-slab floors and relatively symmetric concrete shear walls, on the perimeter and surrounding the elevator core. Damage consisted primarily of shear wall boundary element failure due to the lack of closely-spaced confinement reinforcement. The building was yellow-tagged but was repaired relatively quickly with epoxy injection of the cracks and installation of steel straps at the location of the wall boundary element failures.

Pre-1980 frame buildings typically fared worse than their shear wall counterparts. The Holiday Inn in Van Nuys was a seven-story concrete frame structure built in 1966. The frames consisted of exterior column-spandrels and interior flat slabs. Minor structural damage occurred during the San Fernando earthquake but was repaired using epoxy injection and patches. The building was red-tagged following the Northridge event and required temporary shoring for fear of collapse. The major damage mainly consisted of column shear failure below the fifth floor due to lack of ties (*Figure 2*),

Figure 2. Holiday Inn, Van Nuys. Source: NISEE-PEER, University of California, Berkeley.

which led to significant spalling and buckling of the longitudinal reinforcement. The building was later retrofitted with concrete shear walls.

Champaign Tower is a 15-story concrete building in Santa Monica and experienced extensive perimeter column damage due to deep parapets on the balconies. This "short-column" behavior results from high shear demands that the columns attract due to their high stiffness. The coupling beams in the shear walls in the orthogonal direction also experienced significant shear damage. Surprisingly, the damaged columns were still able to maintain gravity load resistance.

Saint John's Hospital is another building located in Santa Monica and consisted of several buildings, built between 1942 and 1966, that were damaged. The Main Wing and the South Wing were yellow-tagged, and the North Wing was red-tagged and demolished. The North Wing lateral system comprised perimeter punched concrete walls. Significant shear cracking occurred in the piers and spandrels at the second floor. There was less wall at this level than the one above, resulting in a likely weak-story. Additionally, the piers were relatively short in many locations creating a "short-column" condition. Taller piers at the second floor did not experience shear cracking.

Much of the column damage and collapses were caused because the columns, designed for gravity loads only, were not detailed to accommodate the displacements they would undergo during an earthquake. As a result, shear failures occurred and did not provide proper confinement for the longitudinal reinforcement, causing loss of gravity-load carrying capacity. Similarly, damage occurred in frame buildings that utilized flat slab floor construction that was not designed to accommodate large displacements and the shear demands imposed on them at the column. A flat slab building with perimeter moment frames in the Sherman Oaks area is one such building. Slab damage, including spalling and concrete crushing, was observed on the top and underside at the columns around the outline of the drop panel. This building was red-tagged by the city. A few similar failures were also observed in residential concrete podium garages with wood structures above.

Several buildings and parking garages saw partial or total collapse, including the Kaiser Permanente Office Building (*Figure 1*) in Northridge and two garages at the Northridge Fashion Center. The Kaiser collapse was attributed to inadequate confinement in the columns and shotcrete shear walls inadequately attached to the frame. The Northridge parking garages were relatively new structures (circa 1988) but were constructed of precast double and inverted tees. Failure was attributed to large diaphragm movements causing the failure of gravity columns that lacked proper confinement. Additionally, large out-of-plane displacements of the perimeter frames occurred causing the precast beams to unseat. Collector failures were also observed in the topping slabs of precast decks in the vicinity of shear walls.

A large percentage of rigid-wall-flexible-diaphragm buildings were also damaged. These buildings typically comprise walls cast on the ground and tilted up into position with a panelized wood roof system. Several roof collapses occurred due to failures of the connection between the tilt-up panels and diaphragm (See the past STRUCTURE article, April 2019, by Lawson and McCormick).

In general, post-1980 and retrofitted buildings performed as intended with a few exceptions. A retail facility in Topanga Plaza, constructed in the early 1960s, was retrofitted in 1989 through the addition of shotcrete walls. These walls were attached to the existing walls with dowels designed to transfer the calculated seismic forces. The new walls were designed to resist seismic load in tandem with the existing walls. More damage occurred in the new walls; the existing walls exhibited sliding shear failures at the base, transferring most of the load to the new walls. Additionally, many of the cracks in the new walls occurred along horizontal




planes likely caused by shrinkage of the shotcrete walls and the lack of gravity loads on the new walls.

## Recommendations Following the Earthquake

Following the earthquake, significant field investigations and studies were performed by structural engineers that resulted in several recommendations to improve the performance of concrete buildings. The UBC was published every three years, but interim changes are often produced; several changes directly related to the failures observed in Northridge were implemented in the 1996 UBC Supplement.

1) The strength-reduction factor for reinforcement used for diaphragm chords and collectors in topping slabs over precast concrete members was reduced to 0.6 from 0.7.
2) Minimum thickness of topping slabs placed over precast floor and roof elements was increased from 2.5 to 3 inches or 6 times the diameter of the largest slab reinforcing bar.
3) Spacing limits and transverse reinforcement requirements were added for chord and collector reinforcement at splices and anchorage zones.
4) The coupling beam definition was expanded to include all beams connecting walls regardless of the span/depth ratio. Additionally, a maximum shear strength limit of $10\sqrt{f'_c}$ was added, along with the requirement that longitudinal bars be enclosed with transverse reinforcement.
5) Allowance of smaller amounts of reinforcement in compression members with a cross-section larger than required for loading was removed for members in Seismic Zones 3 and 4.
6) Stricter requirements were implemented for frame members not part of the lateral system. Tie spacings were reduced for members with induced moments and shears (from $3(R_w/8)$ times the displacements) that do not exceed the design moment and shear strength of the member. Additionally, when the axial load in those members exceed 30% of the design axial strength, they must be reinforced according to the provisions for lateral frame members.

The City of Los Angeles/SEAOSC Task Force also recommended that DBS survey and identify all concrete structures constructed before 1976 and develop a mandatory retrofit ordinance. As discussed below, the ordinance has finally been implemented albeit 25 years later.

## Current Status

Modern standards such as ASCE 7, *Minimum Design Loads for Buildings and Other Structures*, and ACI 318, *Building Code Requirements for Structural Concrete*, contain the requirements for concrete structures designed for seismic resistance. The information provided represents a vast body of knowledge gained through observations after earthquakes and theoretical and experimental research performed at universities. However, there are still thousands of pre-1980 NDC buildings in high seismic regions in the U.S. and abroad. The California Seismic Safety Commission estimates that there are 40,000 in California. The SEAONC Existing Buildings Committee, in cooperation with EpiCenter, recently completed an inventory based on all Sanborn maps for the City of San Francisco. The inventory resulted in an estimate of 3,400 pre-1980 concrete buildings, verifying the estimate calculated by the Concrete Coalition. The risk of these buildings has not been accurately quantified and is difficult given the variability of building configurations, system types, and a frequent lack of drawings. Methodologies to determine the risk of these buildings include ASCE 41, *Seismic Evaluation and Retrofit of Existing Buildings*, Tiers 1, 2, and 3, and the recently developed methodology for ranking buildings in an inventory, ATC 78, *Seismic Evaluation of Older Concrete Frame, Frame-Wall, and Bearing Wall Buildings for Collapse Potential*. Several Southern California cities have recently adopted ordinances that require owners to assess the collapse potential of their older concrete buildings and retrofit these if the assessment deems this necessary. These cities include the City of Los Angeles, West Hollywood, and Santa Monica. San Francisco is currently deciding what to do about the NDC building stock in the city. The knowledge to design and retrofit concrete buildings safely currently exists. Hopefully, the current stock of NDC buildings will be able to be economically retrofitted before the next big one hits.▪

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