Concrete Parking Structures and the Northridge Earthquake

Performance and Resulting Building Code Changes

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One of the most iconic images of the 1994 Northridge Earthquake is the photograph of a collapsed precast concrete parking structure at California State University, Northridge. While it was only one of many precast parking structures that suffered extensive damage as a result of the earthquake, it illustrates the juxtaposition of the incredible ductility exhibited by the perimeter columns with the collapse of the overall structure. It epitomizes one of the primary performance issues highlighted by the earthquake. The failure of numerous concrete parking structures during the earthquake, both precast and cast-in-place, led to an in-depth examination of the current design practices and ultimately led to several building code changes to improve the performance of these types of structures.

Figure 1. Precast concrete garage failure at Cal State Northridge. Courtesy EERI.

Representative Failures

California State University, Northridge

This partially collapsed garage at California State University was a relatively new, four-level precast concrete garage, approximately 18 months old at the time of the Northridge event. Given the age of the structure, it is likely that it was designed in conformance with the 1991 *Uniform Building Code* (UBC) requirements.

The design of the garage included a perimeter "ductile" concrete lateral force-resisting frame, with the exterior columns designed to carry all the lateral loads and the interior columns designed to carry only the vertical loads. As shown in *Figure 1*, the exterior columns exhibited a significant degree of ductility; however, it is likely that the interior columns, designed for vertical loads only, were unable to accommodate the loads imposed as the structure experienced significant lateral displacement. The failure of vertical-load-only columns in structures where the lateral resistance was concentrated in perimeter lateral load-resisting frames was a valuable lesson learned from the Northridge event and was the impetus for future building code changes.

Northridge Fashion Center

At the Northridge Fashion Center, two large, precast, prestressed concrete garages collapsed (*Figure 2*). The garages were relatively new in that the mall had just opened in 1991. The garage at the southwest corner of the shopping plaza had a vertical load-resisting system comprised of precast concrete columns supporting precast concrete beams, while the lateral load-resisting system was comprised of concrete shear walls in each of the structure's principal directions. There was visual evidence of damage to the precast columns as well as a loss of support for the precast beams. It was interesting to note that that the concrete shear walls, which were likely intended as the primary lateral elements, suffered little if any damage. A garage in the northwest corner of the plaza, with similar construction, failed as well. There were likely several contributing factors to the collapse of the garages. The connections between the precast elements were likely insufficient to allow the elements to maintain continuity as the structure underwent significant displacement. Also, the lack of continuity reinforcement across the construction joints in the concrete slab may have limited the ability of the diaphragms to transfer the loads to the shear walls adequately.

A one-story cast-in-place concrete structure at the Center had damage to the circular concrete columns that supported the concrete drive ramp. The column's transverse reinforcement, which was spaced at 12 inches on-center, was likely inadequate to provide the needed confinement for these columns; the columns were subjected to a high level of shear due to their increased stiffness which can be contributed to their relatively "short" length.

In addition to the practice of utilizing "independent" lateral load-resisting frames, with strength and detailing different from the standard vertical frame elements, there were several other factors that may have contributed to the significant level of damage in the concrete parking structures. These factors include the practice of designing parking structures at a minimum code compliant level, the irregularity of structural systems often found in multi-story parking structures with interior ramps, and the marginal connection of precast elements.

Changes to the Building Code

After the 1994 Northridge Earthquake, there were two code change cycles (1995 and 1996) that provided opportunities to incorporate lessons learned into the 1997 UBC, which was the premier code for seismic design at that time. Most of the lessons applicable to parking structures fell into three basic categories: Deformation Compatibility, Design of Collectors, and Design of Diaphragms.

The online version of this article contains a table that provides the 1997 UBC approved code changes in these three categories, with the reason given for the code change. The information in the table was collected from a variety of resources, including the 1997 *Analysis of Revisions for the Uniform Building Code* published by the International Code Council's legacy organization, the International Conference of Building Officials.

Figure 2. Failure of precast garage at the Northridge Fashion Center. Courtesy EERI.

1997 UBC Section 1633.2.4, Deformation Compatibility, was added because of the deformation-induced damage during the Northridge Earthquake to parking garage elements not part of the lateral force-resisting system. The added language required elements not part of the lateral force-resisting system, regardless of material type, to be designed and detailed to maintain support of the design dead plus live loads when subjected to the expected deformations caused by seismic forces; plus, additional considerations were stipulated. For concrete and masonry lateral force-resisting elements, the assumed flexural and shear stiffness properties were limited to a maximum of one-half the gross section properties unless a rational cracked-section analysis was performed. Also, new design and detailing requirements for concrete were added to Chapter 19 to improve deformation ductility and ensure their ability to continue to support gravity loads. These new provisions were submitted by the SEAOC Seismology Committee (Chair Bob Chittenden) for the 1995 code development cycle. The code change was "approved as revised" by the ICBO Lateral Design Code Development Committee. There were further amendments to the code change approved at ICBO's 1995 Annual Education and Code Development Conference in Las Vegas, Nevada. The Chapter 19 deformation compatibility provisions cited in the table (*online version of this article*) were submitted by the Portland Cement Association (Mark Kluver). This code change was approved by the ICBO Lateral Design Code Development Committee without amendments at their meeting in Des Moines, Iowa, in February 1995.

1997 UBC Section 1633.2.6, Collector Elements, was added because collector elements failed in parking garages during the Northridge Earthquake, and lateral loads were not delivered to the shear walls as intended by design. The new provisions required that collector elements, splices, and their connections to resisting elements be designed to resist forces increased by the new overstrength factor introduced in the 1997 UBC. The overstrength factor was introduced in recognition that forces generated in the lateral force-resisting system can be two to three times the design seismic forces. Failures of collectors in the Northridge Earthquake resulting in disconnection of the building from the lateral force-resisting system and, in some cases, a loss of a portion of the vertical load-carrying system demonstrated these higher design forces were warranted. The collector element code change was submitted during the 1996 code development cycle by Forell/Elsesser Engineers Inc. (Mark Jokerst). The code change was "approved as revised" by the ICBO Lateral Design Code Development Committee with further amendments submitted by the SEAOC Seismology Committee and approved at ICBO's 1996 Annual Education and Code Development Conference in St. Paul, Minnesota.

1997 UBC Section 1921.6.12, Diaphragms, was added because topping slabs over precast concrete members, typically intended to be used as the diaphragm to transfer the lateral loads, performed poorly during the Northridge Earthquake. Minimum thickness requirements were added as well as requirements for mechanical connectors used to transfer forces between the diaphragm and the lateral force-resisting system. The diaphragm code change (*cited in the table in the online version of this article*) was submitted by the California Division of the State Architect (Vilas Mujumdar) and the Portland Cement Association (Mark Kluver). It was approved by the ICBO Lateral Design Code Development Committee without amendments at their meeting in Sparks, Nevada, in February 1996.

Conclusion

The failure of numerous concrete parking garages during the Northridge earthquake highlighted several major issues with the lateral force-resisting systems of these types of structures. In "bare" structures, such as parking garages, the significant lessons learned included the importance of ductile interconnections between the different elements of the lateral force-resisting system, deformation compatibility between "vertical only" elements and the lateral force-resisting system, the importance of designing for ramp and diaphragm discontinuities, and designing non-seismic systems for the full expected seismic drift. Following the 1994 Northridge earthquake, several structural engineering and building code organizations worked together to quickly develop and adopt code modifications to address these issues in future building codes.▪

The online version of this article includes a *Table* referencing several changes to the 1997 UBC resulting from the performance of parking garages in the Northridge earthquake. Also, the online version contains references. Please visit **www.STRUCTUREmag.org**.

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1997 UBC PROVISION	REASON
DEFORMATION COMPATIBILITY	
1633.2.4 Deformation compatibility. All structural framing elements and their connections, not required by design to be part of the lateral-force-resisting system, shall be designed and/or detailed to be adequate to maintain support of design dead plus live loads when subjected to the expected deformations caused by seismic forces. $P\Delta$ effects on such elements shall be considered. Expected deformations shall be determined as the greater of the Maximum Inelastic Response Displacement, ΔM, considering $P\Delta$ effects determined in accordance with Section 1630.9.2 or the deformation induced by a story drift of 0.0025 times the story height. When computing expected deformations, the stiffening effect of those elements not part of the lateral-force-resisting system shall be neglected. For elements not part of the lateral-force-resisting system, the forces inducted by the expected deformation may be considered as ultimate or factored forces. When computing the forces induced by expected deformations, the restraining effect of adjoining rigid structures and nonstructural elements shall be considered and a rational value of member and restraint stiffness shall be used. Inelastic deformations of members and connections may be considered in the evaluation, provided the assumed calculated capacities are consistent with member and connection design and detailing. For concrete and masonry elements that are part of the lateral- force-resisting system, the assumed flexural and shear stiffness properties shall not exceed one half of the gross section properties unless a rational cracked-section analysis is performed. Additional deformations that may result from foundation flexibility and diaphragm deflections shall be considered. For concrete elements not part of the lateral-force-resisting system, see Section 1921.7.	Deformation compatibility provisions have been largely ignored by the design community. In the Northridge earthquake, deformation-induced damage to elements which were not part of the design lateral-force-resisting system resulted in structural collapse. Damage to elements of the lateral-framing system, whose behavior was affected by adjoining rigid elements, was also observed. This has demonstrated a need for stronger and clearer provisions. These changes attempt to emphasize the need for specific design and detailing of elements not part of the lateral system to accommodate expected seismic deformations. Ideally, deformation compatibility would be dealt with by improving design and detailing requirements on a material-by-material basis to improve deformation ductility so as not to place an unrealistic calculation burden on the design engineer. New provisions introduced by ACI and PCA take this approach for concrete systems. However, since such provisions are not in place for all materials, the above changes are provided. More guidance is offered on computation of deformations, including the requirement that "cracked section" stiffness values be used. The 50 percent of gross section value is admittedly conservative, but such conservatism was deemed appropriate by the SEAOC Seismology Committee until further research is performed. The designer can replace these values with a "rational" analysis. Foundation flexibility and diaphragm deformation are highlighted as possible important consideration. It is clarified and emphasized that inelastic action is permitted in the evaluation of members and their connections.
1921.7 Frame Members Not Part of the Lateral Force-Resisting System	This change represents emergency provisions developed by the ACI 318 Committee to address the poor performance of some

1921.7.1 Frame members assumed not to contribute to lateral resistance shall be detailed according to Section 1921.7.2 or 1921.7.3, depending on the magnitude of moments induced in those members when subjected to Δ_M. When induced moments under lateral displacements are not calculated, Section 1921.7.3 shall apply.

1921.7.2 When the induced moments and shears under lateral displacements of Section 1921.7.1 combined with the factored gravity moments and shear loads do not exceed the design moment and shear strength of the frame member, the following conditions shall be satisfied. For this purpose, the load combinations (1.4D + 1.4L) and $0.9D$ shall be used.

1921.7.2.1 Members with factored gravity axial forces not exceeding ($A_g f'_c/10$), shall satisfy Section 1921.3.2.1. Stirrups shall be placed at not more than $d/2$ throughout the length of the member.

1921.7.2.2 Members with factored gravity axial forces exceeding ($A_g f'_c/10$), but not exceeding $0.3P_o$ shall satisfy Sections 1921.4.3, 1921.4.4.1, Item 3, and 1921.4.4.3. Design shear strength shall not be less than the shear associated with the development of nominal moment strengths of the member at each end of the clear span. The maximum longitudinal spacing of ties shall be s_o for the full column height. The spacing s_o shall not be more than (1) 6 diameters of the smallest longitudinal bar enclosed, (2) 16 tie-bar diameters, (3) one-half the least cross-sectional dimension of the column and (4) 6 inches (152 mm).

1921.7.2.3 Members with factored gravity axial forces exceeding $0.3P_o$ shall satisfy Sections 1921.4.4 and 1921.4.5.

1921.7.3 When the induced moments under lateral displacements of Section 1921.7.1 exceed the design moment strength of the frame member, or where induced moments are not calculated, the following conditions in Sections 1921.7.3.1 through 1921.7.3.3 shall be satisfied.

1921.7.3.1 Materials shall satisfy Sections 1921.2.4, 1921.2.5 and 1921.2.6.

1921.7.3.2 Members with factored gravity axial forces not exceeding ($A_g f'_c/10$) shall satisfy Sections 1921.3.2.1 and 1921.3.4. Stirrups shall be placed at not more than $d/2$ throughout the length of the member.

concrete buildings in the Northridge earthquake. The modifications significantly increase the amount of transverse reinforcement in members not part of the lateral force-resisting system in high seismic risk areas. The detailing requirements imposed on members that are part of the lateral force-resisting system provide that the members may undergo deformations that exceed the elastic capacity of the member without significant loss of strength. Members that are not part of the designated lateral force-resisting system are not required to meet all the detailing requirements of members that are relied on to resist lateral forces, but they must be able to resist deformations above the service level and still be able to support gravity loads. These revisions recognize that actual displacements resulting from earthquake forces may be several times larger than displacements calculated using the code-specified design forces and commonly used analysis models. Section 1921.7.1 defines a nominal displacement amplitude for the purpose of setting detailing requirements. Actual displacements may exceed the value listed in Section 1921.7.1. In Section 1921.7.1, the Δ_M notation from Section 1628 is included. Section 1921.7.2 defines details intended to provide a gravity load-carrying system capable of sustaining gravity loads under moderate excursions into the inelastic range. Section 1921.7.3 defines details intended to provide a gravity load-carrying system capable of sustaining gravity loads under more significant inelastic displacements.

1921.7.3.3 Members with factored gravity axial forces exceeding ($A_g f'_c$ /10) shall satisfy Sections 1921.4.4, 1921.4.5 and 1921.5.2.1.	
COLLECTOR DESIGN	
1633.2.6 Collector elements. Collector elements shall be provided that are capable of transferring the seismic forces originating in other portions of the structure to the element providing the resistance to those forces. Collector elements, splices and their connections to resisting elements shall resist the forces determined in accordance with Formula (33-1). In addition, collector elements, splices, and their connections to resisting elements shall have the design strength to resist the combined loads resulting from the special seismic load of Section 1612.4. **EXCEPTION:** In structures, or portions thereof, braced entirely by light-frame wood shear walls or light-frame steel and wood structural panel shear wall systems, collector elements, splices and connections to resisting elements need only be designed to resist forces in accordance with Formula (33-1). The quantity E_M need not exceed the maximum force that can be transferred to the collector by the diaphragm and other elements of the lateral-force-resisting system. For Allowable Stress Design, the design strength may be determined using an allowable stress increase of 1.7 and a resistance factor, ϕ, of 1.0. This increase shall not be combined with the one-third stress increase permitted by Section 1612.3, but may be combined with the duration of load increase permitted in Division III of Chapter 23.	In the 1994 Northridge Earthquake, collector elements failed in parking garages, and loads were not delivered to the shear walls as intended by design. Overstrength factors were added to the 1997 UBC in response to Northridge Earthquake damage. It is understood that during the design basis ground motion, the forces generated in the lateral-force-resisting system will be approximately two to three times the design seismic forces. This inherent "system overstrength" can cause failures in the collector system if comparable overstrength in the collector system is not provided. Failures of collectors can result in disconnection of the building from this lateral-force-resisting system, or if the collector system also supports vertical loads, can result in a loss of a portion of the vertical load-carrying system. Because the system overstrength is unavoidable, this level of strength in the collector system is warranted. This provision will help to ensure that inelastic energy dissipation occurs in the ductile lateral-force-resisting elements (frames, braces, walls), rather than in the collectors and connections. The added exception recognizes that the design provisions would result in larger collectors and a large increase in the quantities of fasteners. This could make construction difficult and actually decrease the capacity because of splitting of the wood member from large quantities of fasteners. Light-frame systems generally include a number of lines of resistance carrying fairly low loads when compared to other types of construction, and damage to properly designed and constructed wood collector elements have not been reported. Hence,

	requiring higher design force levels for light-frame collectors does not seem necessary.
DIAPHRAGM DESIGN	
1921.6.12 Diaphragms. Diaphragms used to resist prescribed lateral forces shall comply with the following: 1. Thickness shall not be less than 2 inches (51 mm). 2. When mechanical connectors are used to transfer forces between the diaphragm and the lateral system, the anchorage shall be adequate to develop 1.4 A_s f_y, where A_s is the connector's cross-sectional area. 3. Collector and boundary elements in topping slabs placed over precast floor and roof elements shall not be less than 3 inches (76 mm) or 6 d_b thick, where d_b is the diameter of the largest reinforcement in the topping slab. 4. Prestressing tendons shall not be used as primary reinforcement in boundaries and collector elements of structural diaphragms. Precompression from unbonded tendons may be used to resist diaphragm forces.	Thin topping slabs over precast concrete members performed poorly during the January 1994 Northridge earthquake. The slabs had excessive deformations because of low stiffness. The change incorporated additional diaphragm provisions in an organized fashion in this section. In general, for precast construction, a system of mechanical connectors is needed to ensure transfer of diaphragm forces to the seismic-force-resisting system. Regardless of the positive connection system used, ties are needed to ensure that elements do not drop off supports during severe seismic excitations. Further, continuous load paths must be established to adequately transfer the lateral loads to the foundation. Mechanical connectors consist of two basic parts: an embedment that provides anchorage for the connection and a connector that is the element crossing the interface. The embedment must be markedly stronger than the probable connector capacity. Thus, the embedment is required to develop $1.4A_sf_y$ where A_s is the connector's area to ensure that any yielding that takes place will occur in the body of the connection. Large in-plane forces and bending moments in diaphragms may cause cracks, which can result in yielding of the diaphragm reinforcing steel as well as large axial compressive forces in the diaphragm boundary elements and frame members, which resist flexural and axial forces induced by the earthquake loads.