Performance of Precast/ Prestressed Concrete Building Structures During Northridge Earthquake

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Observations are reported of the performance in the January 17, 1994, Northridge, California, earthquake of buildings in which precast/prestressed concrete components were utilized. Ground motions recorded during the earthquake are discussed in relation to fault movements, observed responses at building sites, and prevailing building code design requirements. The results of field observations of parking garages, cladding for buildings and foundations for residences are reviewed. For garages, the performance of structures utilizing primarily precast/prestressed concrete components is compared with that of garages utilizing other materials. It is concluded that, with the exception of buildings in the immediate vicinity of the epicenter, engineered structures, including those with precast/prestressed concrete components, generally performed well. Parking garages, particularly those with large plan areas, did not perform as well as other types of buildings. The greatest damage was in collector elements funneling lateral forces to the vertical elements of the lateral load resisting system and in the columns of gravity load carrying systems that were not intended to be part of the seismic resisting system. No damage was observed to cladding due either to inadequacies of the precast components or their connections to the building's structural system.

The January 17, 1994, Northridge, California, earthquake caused extensive damage to many structures within a 25 mile (40 km) radius of its epicenter located as indicated in Fig. 1 by a star. That epicenter was approximately 1 mile (1.6 km) south and 11 miles (18 km) below the surface of the Northridge Fashion Center. The earthquake registered a magnitude of 6.8 on the Richter scale. With some notable exceptions, the majority of engineered structures within the affected region performed well, including structures with precast concrete components.

In Southern California, the three major applications of precast concrete are in components for parking garages, cladding for buildings, and foundations for multi-family residences. This report reviews the performance of precast concrete for those three areas. As a class, parking garages, including several of precast concrete construction, appeared to suffer more damage than other buildings. Therefore, this review provides information on the performance of garages with precast components as well as that of garages with other structural systems.

Fig. 1. Peak horizontal and vertical accelerations as measured at different locations by CSMIP and locations of parking structures visited.

GROUND MOTIONS

There were no strong motion instruments located in the immediate vicinity of the Northridge Fashion Center. Therefore, the ground motions to which structures at that Center and at the nearby California State University-Northridge campus were subjected to can only be hypothesized. Fig. 1 is a map, taken from the "Fifth Quick Report" by the California Strong Motion Instrumentation Program (CSMIP),[1] that shows where instruments were located and the peak horizontal and vertical accelerations they recorded.

The closest free field instrument for which accelerograms were recorded was located on soil at Tarzana-Cedar Hills Nursery, approximately 4½ miles (7 km) south of the epicenter. Those records showed peak horizontal and vertical accelerations of 1.82*g* and 1.18*g*, respectively, a period of about 0.3 to 0.5 seconds and a duration of strong ground shaking of about 20 seconds.

Because records are affected by soil conditions, the Tarzana record is not as likely to be representative of the motions to which the Center and campus structures were subjected as those from instruments located northeast of the epicenter and shown in Fig. 2. The closest record was from the base of a seven-story hotel in Van Nuys, 4 miles (6.5 km) northeast of the epicenter. It showed peak accelerations of 0.47*g* east-west and 0.41*g* north-south horizontally and 0.30*g* vertically, a period of about 0.3 seconds and a duration of strong ground shaking of about 20 seconds.

At the Arleta-Nordhoff Ave Fire Station 5½ miles (9 km) northeast of the epicenter, peak accelerations were 0.35*g* east-west and 0.29*g* north-south horizontally and 0.59*g* vertically. The period was about 0.3 seconds and the duration of strong ground shaking about 20 seconds. At the Sepulveda V.A. Hospital, about 5 miles (8 km) northeast, peak accelerations were 0.94*g* north-south and 0.74*g* east-west horizontally and 0.48*g* vertically.[2] At Sherman Oaks, about 6 miles (10 km) southeast of the epicenter, at a 13-story commercial building, recorded peak accelerations in the second sublevel were 0.46*g* north-south and 0.24*g* east-west horizontally and 0.18*g* vertically. The period was about 0.4 seconds and the duration of strong ground shaking about 16 seconds.

There is reason to expect that local ground motions were dependent on focusing effects as well as soil conditions. Focusing effects are usually near field effects believed to be due to an interaction of seismic waves reflected from the irregular configuration of the underlying bedrock. When the crests of criss-crossing waves intersect, motions are amplified. The soil can also filter and amplify the characteristics of the motion resulting in motions that are larger and have different predominant periods than those of the bedrock motion. The Northridge

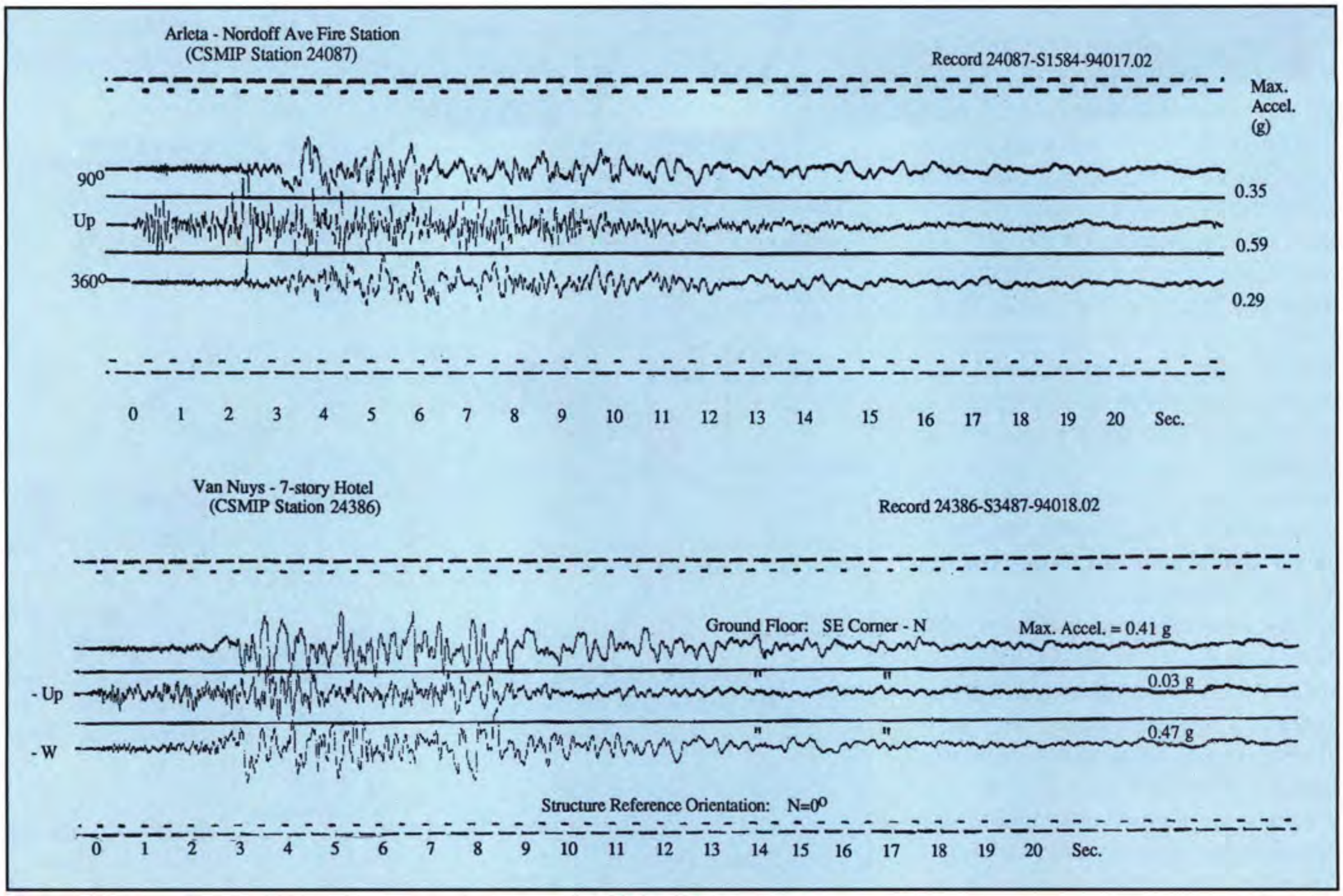

Fig. 2. Typical CSMIP earthquake motion records: Arleta-Nordhoff Ave Fire Station and Van Nuys seven-story hotel.

Fig. 3. Partially collapsed southwest garage at Northridge Fashion Center (Garage 1). (Courtesy: *Los Angeles Times*).

records show both near field effects and soil amplification effects.

The fault identified as causing this earthquake was an unknown, buried, thrust fault. Movements on such faults cause high vertical accelerations for the ground above them. The term buried is used because there has been no evidence at or near the ground surface of the fault's existence. Geologists estimate that the Los Angeles Basin may contain many such faults with planes essentially parallel to the ground's surface. The evidence from this earthquake of such faults' potential to do damage complicates both the prediction of earthquake activity in the Basin and the development of seismic design regulations for structures located above such faults.

Vertical motions are caused primarily by *p*-waves that propagate directly out from the epicenter at about 1 mile per second (1.6 km/s), depending on the geology and soil conditions. Horizontal motions are caused by *s*-waves that move perpendicular to *p*-waves and at about half the speed. The effects of *p*-waves are usually less damaging than those of *s*-waves but this Northridge earthquake may demonstrate an exception.

The vertical motions in the Northridge records often differed significantly from those normally anticipated:

First, the magnitude of those motions, compared to the horizontal motions, was large.

Second, the duration of strong ground shaking associated with those

Fig. 4. North face of northwest garage at Northridge Fashion Center with partially collapsed area at left (Garage 2).

Fig. 5. Collapse of second-story portion of precast concrete column on south face of northwest garage at Northridge Fashion Mall (Garage 2).

Fig. 6. Collapsed northeast corner of northwest garage of Northridge Fashion Center showing precast concrete column, spandrel panel and floor details (Garage 2).

vertical motions was large. Even though that duration was only about half the duration for horizontal motions, there were multiple peak accelerations throughout that time period close to the maximum vertical acceleration peak.

Third, where there were strong vertical motions, the vibrations resulting from those motions began about 3 seconds prior to the initiation of strong horizontal motions.

As a result, structures with large plan areas, such as many parking garages, would develop significant vertical vibrations before the arrival of the horizontal motions.

GROUND MOTIONS AND BUILDING CODE PROVISIONS

Structures in the Northridge area are designed to resist forces mandated by the provisions of the Los Angeles Building Code which, in general, follows the seismic provisions of the Uniform Building Code (UBC).[3] The provisions of the UBC are updated every 3 years so that a structure designed to the 1991 UBC may end up with properties that differ from those for the same structure designed to an earlier version of UBC. In general, seismic design provisions have become more rigorous as our understanding of the factors affecting seismic risk has increased. Further, UBC requires design for horizontal earthquake forces only.

For the Northridge area, seismic forces in recent UBCs have depended on the anticipated effective peak horizontal acceleration, Z; a building period dependent seismic response coefficient, C_{ts}, that included the influence of soil type; the importance of the building I; a force reduction coefficient, R_w, dependent on the seismic resisting system used for the building; and the building's weight.

In accordance with the 1988 UBC, for structures on stiff soils, the value of C_{ts} remained constant at 2.75 for building periods up to 0.3 seconds and then decreased with increasing building period. The Z value was $0.4g$ and the R_w value was 6 for concrete shear walls and 12 for special moment resisting concrete space frames. Thus, for a typical two- or three-story parking structure with shear walls as the lateral load resisting system, properties are $C_{ts} = 2.75$, $I = 1.0$, $R_w = 6$ and $Z = 0.4g$. The resultant ratio of the design base shear to the building's weight is 0.18.

For accelerations greater than $0.4g$, it was to be expected that buildings designed according to the 1988 UBC would be damaged structurally. However, collapse was not to be expected unless the effective peak accelerations exceeded $0.85g$.[4]

Since the effective peak horizontal accelerations in the epicentral area, with the exception of the Tarzana, Santa

Table 1. Parking garages visited (January 1994).

Garage number	Location; Damage	Lateral load system; Gravity load system; Supported floors; Approximate plan area	Estimated date built
1	Northridge Fashion Center South Extensive collapse	Two 30 ft shear walls north-south and east-west and cast-in-place topping; Precast columns, inverted tee and double tees; Two; 240 ft east-west x 330 ft north-south	1988
2	Northridge Fashion Center North Collapse northeast quadrant and column sheared at south end of east north-south shear wall line	Similar to Garage 1 200 ft north-south x 400 ft east-west	1987
3	Northridge Fashion Center West 8 in. movements, damaged pedestrian access ramps, crushing at column tops	Special moment resisting cast-in-place reinforced concrete frames; One-way slabs over beams and spiral columns; Below grade; 400 x 400 ft	Pre-1988
4	Northridge Fashion Center East 8 in. movements	Duplicate of Garage 3	
5	California State University, Northridge Collapse east end and partial collapse west end	Special moment resistant exterior frames for eight central bays north-south and east-west; Site precast exterior end frames and interior columns, plant cast beams; One-way post-tensioned slabs east-west except two-way for end bays; Three; 400 ft east-west x 324 ft north-south	1991
6	Glendale Fashion Center Crushing of short exterior columns, fracture of diaphragm connection to north-south walls, slip second floor north-south wall construction joint	Seven 15 ft shear walls north-south; Four 1 ft plus one 120 ft shear wall east-west; Cast-in-place columns and east-west beams with composite; double tee floors; Two and one-half; 240 ft east-west x 340 ft north-south	Pre-1984
7	Glendale Civic Center, Broadway & Wilson Collapse of end of fourth floor down to grade Failure of top diaphragm	Three 40 ft shear walls north-south; Two 60 ft shear walls east-west cast-in-place; Precast columns, inverted tees and composite double tee floor; Three increased recently to three and a half; 180 ft north-south x 120 ft east-west	1988
8	Glendale Galleria, Colorado and Columbus Drop of floor 4 to 8 in. in places due to damage to pedestals	Ductile frames made by anchoring Conrad Table legs to pedestals; Conrad Tables with cast-in-place topping and connecting slabs; Two; 1000 ft north-south x 300 ft east-west	Pre-1984
9	Sherman Oaks Galleria East Garage, Woodman and Riverside North-south shear wall joint fracture, slight north-south column connection damage	Two 40 ft north-south shear walls/structural Two 40 ft east-west shear walls/structural Ordinary frames with post-tensioned beams north-south One-way slab east-west; One; 300 ft north-south x 240 ft x two structures east-west	
10	Sherman Oaks Galleria South Garage Partial collapse west structure. Crushing and fracture of columns, shear wall joint fracture west structure; shear wall spread footing uplifts east and central structure	Two 40 ft shear walls/structure north-south and east-west; Cast-in-place columns and post-tensioned beams north-south and one-way slab east-west; Two; 180 ft north-south x 240 ft x three structures east-west	
11	Ventura & Sepulveda Blvds., southeast corner Column and precast spandrel cosmetic damage from pounding, column shear failure over entry, loss of one precast spandrel	North-south and east-west shear walls; Cast-in-place columns and post-tensioned east-west beams; Eight; 200 ft north-south x 120 ft east-west	1988
12	Ventura & Sepulveda Blvds, southwest corner No damage to steel frames; fracture of masonry walls; garage in use	North-south and east-west masonry shear walls; Steel simple frame with metal deck composite floors; Three	1960s

Table 1 (cont.). Parking garages visited (January 1994).

Garage number	Location; Damage	Lateral load system; Gravity load system; Supported floors; Approximate plan area	Estimated date built
13	Kaiser Permanente East Garage La Cieniga & Cadillac Internal collapse; This structure was seen in partially demolished state	Special moment resisting frames east-west; Exterior shear walls north-south; Cast-in-place columns and post-tensioned beams; Five; 120 ft east-west x 150 ft north-south	1990
14	Kaiser Permanente West Garage La Cieniga & Cadillac Uplift of north-south shear wall footings, tilt of east stairwell, column damage at mid-height joint	Two 120 ft shear walls north-south and east-west; Cast-in-place columns and post-tensioned beams on corbels, one-way slab east-west; Four-Five; 360 ft north-south x 360 ft east-west; Did not gain access to site	1985
15	Kaiser Permanente, Woodland Hills Cosmetic damage to interior columns, significant damage to exterior gravity columns of south ramp	Shear walls east-west, special moment frames north-south; Cast-in-place columns, post-tensioned beams and one-way post-tensioned slab; Four	1987 1990 1993
16	Hilton Hotel, Woodland Hills Loss of mechanical room at top of elevator shaft, significant damage to masonry of stairwell	Shear walls north-south and east-west; Cast-in-place columns, post-tensioned beams and one-way post-tensioned slab; Eight	1992
17	St. John's Hospital, Santa Monica Severe shear cracking east-west walls and joint slip	Shear walls north-south and east-west; Cast-in-place columns, post-tensioned beams, and one-way post-tensioned slab; Three	1988
18	St. John's Hospital, Santa Monica Buckling of braces, fracture of columns	Braced steel frames; Simple steel frames; Three	1970
19	St. John's Hospital, Santa Monica Cosmetic damage to spiral columns	Cast-in-place shear walls north-south, cast-in-place ductile frame east-west; Three	1960
20	Litton Industries, 19867 Prairie, Chatsworth Minor cracks; in use	Cast-in-place shear walls; Double tees and precast columns; Three	1990
21	Coldwell-Banker, Balboa & Germain Granada Hills; Spalled haunches and masonry wall cracks, displaced precast panel	Cast-in-place masonry shear walls; Double tees and precast columns; Four	1993
22	Devon Industries, 9540 Desoto, Chatsworth Masonry wall collapse dropping double tees	Masonry shear walls; Double tees and precast columns; One	1985
23	Northpark Office Center, 9151 Eton, Chatsworth No problems, in use	Cast-in-place shear walls; Double tees and precast columns; Two	1985
24	Hamer Toyota, Sepulveda & Brand, Mission Hills Collapse of untopped top floor	Masonry and cast-in-place shear walls; Double tees and precast columns; Three	Under construction
25	Glendale Galleria, Central Ave & Broadway Glendale No problems, in use	Cast-in-place shear walls and frame; Double tees and precast columns; Five	1970s
26	Reseda Business Park, 6925 Canby, Reseda Minor spalling, in use	Cast-in-place and masonry shear walls; Double tees and precast columns; One	1990
27	Braemer Country Club, 4001 Reseda, Tarzana No problems, in use	Cast-in-place shear walls; Double tees and precast columns; One	1992
28	Auto Stiegler, 16721 Ventura, Encino Minor cracking, in use	Cast-in-place shear walls; Double tees and high channel beams and precast columns; Five	1992
29	Miller Infinity, 5455 Van Nuys, Van Nuys Minor cracking and spalling in masonry wall, in use	Masonry and cast-in-place shear walls; Double tees and precast columns; Three	1990
30	Van Nuys Retail, Valerio & Van Nuys, Van Nuys No problems, in use	Masonry and cast-in place walls, double tees and precast columns; Two	1989

Fig. 7. Partially collapsed garage at CSU-Northridge looking north (Garage 5). (Courtesy: *Engineering News-Record*).

Monica, and Sepulveda Hospital records, were less than 0.85*g*, buildings designed according to the 1988 UBC should have performed satisfactorily for the horizontal motions of the Northridge earthquake provided the structure could develop the degree of ductility associated with the R_w value used by the designer. The field results, however, suggest that the destructive potential of this earthquake was greater than the potential assumed by the building code. The reasons for that difference could be due to effects caused by the vertical motions or by interactions of vertical and horizontal motion effects.

FIELD OBSERVATIONS

In the week following the earthquake, the investigators separately visited the Northridge area and made visual examinations of the damage to building structures with particular emphasis on those using precast concrete components. In the following, a brief description is provided of some of the characteristics observed for the buildings visited and the extent of the apparent damage, if any. In most cases, access to the site was restricted and in only two cases have the plans for the structures been seen. The information provided here is based on what the investigators could observe and some findings could change as more information becomes available.

PARKING GARAGES

Table 1 is a summary of the garages visited, the damage observed and, where available, the apparent structural characteristics of the garages. Column 2 lists the location of the garage and any damage observed. Garage locations are also indicated spatially on Fig. 1 with colored dots containing numerals that correspond to the listing of the garage in Table 1.

The color for each dot in Fig. 1 is the same as the tag that had been placed on that structure in conformance with ATC-20 procedures.[5] Thus, red indicates a building to which entry was prohibited, yellow one where entry could be made with caution, and green one for which there were no restrictions as a result of earthquake damage. If the garage was a composite of several similar structures, such as Sherman Oaks Galleria-South Garage, the color used for the dot is that for the worst damage.

Column 3 of Table 1 lists in turn the characteristics of the lateral load resisting system for the garage, the gravity load resisting system, the number of supported floors above grade and the approximate overall plan dimensions of the structure. Column 4 lists the date of construction, if known, or the approximate date of construction where there was some evidence to indicate that date.

Extensive partial collapses occurred for two prefabricated, two-story garages at the Northridge Fashion Center and for a 1991 three-story partially site precast garage at CSU-Northridge. Thirty garages, including all those known to have been damaged, within a 20 mile (32 km) radius of the epicenter were visited. Nine of those garages had suffered partial collapse of the parking structure. Thirteen of the garages had suffered apparently repairable damage. Nine garages, utilizing precast concrete components and within 4 miles (6.5 km) of the epicenter, had very minor or no damage. Numerous photographs were taken of

Fig. 8. Northwest corner of CSU-Northridge garage showing gravity load column trees separating from each other as diaphragm fails (Garage 5).

Fig. 9. Gravity load interior columns, southwest corner, CSU-Northridge garage showing crushing of column bases and incipient failure of floor due to loss of support for transfer beams (Garage 5).

Fig. 10. Undamaged Litton Industries garage, Chatsworth, about 1 mile (1.6 km) east of epicenter (Garage 20).

Fig. 11. Coldwell Banker garage, Granada Hills, with mildly damaged column haunches and cracked filler walls about 3 miles (5 km) east of epicenter (Garage 21).

the damage. Most undamaged garages were found to be smaller in extent and to apparently have a greater proportional area of, and more uniformly distributed, lateral load resisting elements than the heavily damaged garages.

The partially collapsed concrete garages with precast components had structural characteristics that varied widely from almost totally prefabricated to totally site cast elements. However, all involved prestressed construction of some type either pretensioning or post-tensioning. Further, most collapses appeared to have been precipitated by failure of the gravity, rather than the lateral, load resisting system.

The city of Los Angeles specifies that the plans submitted by the engineer of record must be buildable. That requirement can be satisfied by showing member sizes and typical details. Thus, framing members intended to resist seismic forces can be designed by a licensed engineer different from the licensed engineer who designs the framing members intended to resist gravity forces. A precast concrete products supplier can do the detailing of his products for gravity loads and have those drawings signed off by his own engineer, provided the plans of the engineer of record show typical connection details.

Codes require that gravity load elements must be able to displace laterally, without collapse, to the same displacements as those expected for the lateral load resisting system. The issue can become which of the two licensed engineers assumed the responsibility for ensuring that condition was satisfied. In many of the garages that had partial collapses, it appears that while some of the lateral load resisting elements were able to take those displacements, the gravity load elements could not and collapsed. This effect was also probably compounded by the earthquake's relatively large vertical accelerations.

One characteristic feature of several of the prefabricated and site precast garages that collapsed was an apparent lack of adequate ties connecting precast floor elements to one another and to their lateral load resisting system. In at least three of the garages, it appeared that failure of the inadequate ties permitted precast members to fall from the upper levels, causing collapse of the lower levels.

Possible inadequacies in the distribution of the lateral load resisting elements throughout the plan of the structure, or the inability of the gravity load resisting system to undergo the lateral displacements permitted by the movements of the lateral load resisting system, were also noted in garages not containing precast concrete components. One other common structural system for garages in the Los Angeles area utilizes a cast-in-place gravity load frame with concrete girders post-tensioned in one direction and a one-way slab post-tensioned in the orthogonal direction. Those frames are cast first and then stiffened laterally by cast-in-place shear walls. It is difficult to make a good connection between the shear wall and the soffit of the post-tensioned beam and, in particular, to properly roughen or grout that con-

Fig. 12. Auto Steigler Parking, Encino. Only minor cracking occurred (Garage 28).

nection. Several failures of that connection were noted.

Garages Within 5 Miles (8 km) of the Epicenter

Shown in Fig. 3 is a *Los Angeles Times* photograph of the partially collapsed Northridge Fashion Center South Garage, Garage 1 of Table 1, on the morning after the earthquake. Shown in Fig. 4 is the north face of a similar garage (Garage 2) which suffered a partial collapse in the northeast corner only (left side of the photograph). Lateral movements of the gravity load column have caused vertical cracking over the depth of the spandrel panel at the thinner cutout section where it crosses that column at the first supported floor.


Fig. 13. Glendale Fashion Center (southwest corner). Lower story columns on right have shortened markedly compared to upper story columns (Garage 6).


Fig. 15. Glendale Fashion Center. Interior view along north face shows double tee floor and column cracking/bar buckling along north face (Garage 6).


Fig. 14. Glendale Fashion Center. Note "short column" cracking on west face (Garage 6).


Fig. 16. Glendale Fashion Center. Note failure of shear wall construction joint and of connection of diaphragm to shear wall in northwest corner of second supported level (Garage 6).

Fig. 17. Glendale Civic Center Parking. Undamaged west face (Garage 7).

Fig. 18. Glendale Civic Center. Collapsed roof and floor areas on east side (Garage 7).

Similar cracks were visible for nearly every north-south column line. Shown in Fig. 5 is the sheared second-story column on the south end of the gravity column and beam system on the north-south shear wall line at the eastern end of the garage. The inserts that attached the upper level spandrel panels to the sides of the column can be seen to have pulled out of the panels. Shown in Fig. 6 is the collapsed northeast quadrant of Garage 2.

In contrast to the conditions for Garage 1, the composite double tees in Garage 2 spanned in the long direction of the building. The east ends of the double tees were supported on a ledge projecting inwards from the bottom of the spandrel. The west ends were supported on a ledger beam, that spanned north-south from column to column. At the exterior column, seats were

Fig. 19. Glendale Civic Center. Fracture line extending across diaphragm to area of crushing and bar buckling on west side (Garage 7).

Fig. 20. Glendale Galleria, Columbus Avenue Parking. Roof level shows precast concrete "Conrad tables" and settlement at joint line (Garage 8).

Fig. 21. Glendale Galleria, Columbus Avenue Parking. Spalling of cast-in-place pedestals caused settlement of tables (Garage 8).

Fig. 22. Glendale Galleria, Central Avenue Parking. Note undamaged garage (Garage 25).

Fig. 23. Garage at southeast corner of Sepulveda and Ventura Boulevards. Damage is due to inadequate separation of precast cladding from columns (Garage 11).

provided for both the ledger beam and the spandrel beams by enlarging the column on three faces while keeping the outside face planar.

The inner columns had haunches to pick up the ends of the ledger beams but otherwise had constant cross sections over their heights. The torsional stability and shear strength of the spandrel beams and the shear strength of the exterior columns above the enlarged lower portion are among the issues that deserve further examination given the high vertical motions experienced in this earthquake.

Shown in Fig. 7 is the partially collapsed Garage 5 photographed looking north. The garage was supported on spread footings. The building's structural system consisted of site-cast precast interior columns with haunches that supported plant-cast prestressed concrete girders spanning to site-cast exterior column-spandrel trees with haunches on their interior faces.

At the center of the exterior face of the garage, at selected trees, concrete was cast around reinforcement protruding from the spandrels to tie the trees together and create a continuous special moment frame to resist horizontal forces. On the upper level, the spandrels of eight trees at the center of each face were tied together. Additional spandrels were tied together at lower levels creating a wedding cake vertical elevation for the exterior special moment frames.

The column trees at the corners were not interconnected to one another except by the slabs. The slabs were cast-in-place and prestressed longitudinally in the east-west direction except at the east and west ends of the structure where two-way prestressing was used to hold the unconnected trees in position. The spandrel beams were faced with brick creating a 2500-car garage with an attractive appearance.

Fig. 8 shows the northwest corner of the garage. The non-connected spandrels of the column trees nearest the corner are separating from one another at midlength. The connected spandrels on the right have functioned as intended with hinging in the spandrels at the column faces.

The cause of the collapse appeared to be failure of the interior gravity load

Fig. 24. Sherman Oaks Galleria-East Garage. South face shows form of garage and cosmetic damage at exterior beam-to-column connections (Garage 9).

Fig. 25. Sherman Oaks Galleria-East Garage. East face shows failure at shear wall to frame connection and yellow tagging of structure (Garage 9).

columns, particularly those in the corners of the structure, in combined compression and bending. Shown in Fig. 9 is the condition of those columns in the partially collapsed southwest corner of the garage. As the bottom of those columns disintegrated, the beams supported by them rotated vertically downward, dropping from the columns' haunches and causing the structure to sag inwards. The slabs, being tied directly to the exterior column trees, pulled those trees inward, perpendicular to their strong axis. The movements were large enough that the unbonded tendons of the slabs often fractured. In Fig. 8, many anchors from all three floors can be seen to be missing. They

Fig. 26. Sherman Oaks Galleria-South Garage. Note fracture of exterior column in northwest corner of gravity load frame (Garage 10).

Fig. 27. Sherman Oaks Galleria-South Garage. Note damage to beam at connection to interior shear wall on spread footing (Garage 10).

Fig. 28. Kaiser Permanente, West Los Angeles-East Garage. Shows a view of the collapsed structure looking west in a direction parallel to the frames (Garage 13). (Courtesy: Portland Cement Association).

were found at distances of up to 90 ft (27 m) from the structure.

Shown in Fig. 10 is the almost undamaged garage (Garage 20) located about 1 mile (1.6 km) east of the epicenter. This garage had cast-in-place shear walls for the lateral load resisting system and double tees supported on ledger beams and precast columns for the three supported floors of the gravity load system. Shown in Fig. 11 is the mildly damaged Garage 21 located about 3 miles (5 km) east of the epicenter. It had structural systems with characteristics very similar to those of Garage 20. There were four supported floors of parking and damage consisted of spalling or fracturing of the cast-in-place column haunches, cracking of the concrete filler walls and loosening of an exterior panel.

Other garages with precast concrete components, in close proximity to the epicenter, that were visited were Garages 22, 23, 24, 25, 26, 27, 28 and 30. One of those garages (Garage 28) is shown in Fig. 12. That garage was located close to where the very strong CSMIP Tarzana record was obtained. Yet the structure suffered only minor cracking and a few spalls in the cladding. The garage was fully operational after the earthquake.

Of the other structures, only Garages 22 and 24 suffered significant damage. Garage 22, containing double tees, had roof top parking built in 1985. The masonry wall that had supported the tees was tall and quite slender for such purposes and its movement, or collapse, probably contributed to loss of support for the tees.

Garage 24 was being erected at the time of the earthquake. While most of the structure was complete, large areas of the top floor had not yet received the cast-in-place topping and curbs that were to tie the floor together and tie the diaphragms into the shear walls. Collapse of the top level resulted. However, the floor below was able to support the debris with only minor cracking. Repair and completion was underway one week after the earthquake.

There were five other areas more re-

Fig. 29a. Kaiser Permanente, West Los Angeles-West Garage. Shows damage to exterior columns on north face (Garage 14).

Fig. 29b. Kaiser Permanente, West Los Angeles-West Garage. Note uplift of spread footing of east shear wall and separation of stairwell tower from shear wall (Garage 14).

Fig. 30. St. John's Hospital, Santa Monica. Fracture and buckling of braced steel frame of center garage (Garage 18).

mote from the epicenter in which damage to buildings was concentrated. Those areas were Sherman Oaks, Glendale, West Los Angeles, Santa Monica and Woodland Hills. Garages in all five areas were damaged (some heavily), including some garages with precast concrete components.

Glendale Garages — 15 Miles (24 km) From Epicenter

In Glendale, four garages (Garages 6, 7, 8 and 25) were examined. The southwest corner of Garage 6 is shown in Fig. 13. This three-level structure had cast-in-place exterior walls and columns, and double tees connected to precast ledger beams supported on circular cast-in-place columns and a central zig-zag shear wall with an east-west longitudinal axis. Heavy exterior, masonry faced, spandrels converted the exterior columns into "short columns" that fractured and crushed. In Fig. 13 the lower story columns on the right hand (south) face of the building have shortened markedly by comparison to the upper story columns of the same face.

In Fig. 14, a column on the west face of the building shows typical "short column" diagonal cracking at its top increasing in intensity and transitioning to crushing at its bottom. An interior view of the columns along the north face is shown in Fig. 15. The spandrels framed only part of the cross section of the column, and column ties were inadequate allowing the vertical bars to buckle over the full height of the column after loss of concrete cover.

Shown in Fig. 16 are conditions in the northwest corner of the second supported floor where it was connected to the shear wall. The construction joint in the wall immediately above that level had slipped and the connection of the topping to the wall had failed. At that location, the longitudinal axis of the double tee was parallel to the longitudinal axis of the wall.

The interior columns and shear wall of the building were able to keep it from swaying. The crushed short exterior columns settled straight down. Although the building was badly damaged on its exterior, it should be repairable.

Shown in Fig. 17 is Garage 7 which provided parking for Glendale city employees and the Glendale police. This structure at one time had two and one-half supported floors but that had been increased recently to three floors with an addition to the east of the building. That addition was connected to a newly completed civic building by an independently supported pedestrian walkway.

The form of the structure can be seen from Figs. 17 and 18, which show that lateral resistance was provided by cast-in-place shear walls and a gravity load system by precast columns and double tees supported on ledger beams spanning between those columns. There were also heavy cladding panels that often carried planter boxes. The pedestrian walkway is on the left in Fig. 18.

The terminal upper end of the parking extended to a precast column connected by welded bolts to the top of the column in the center of Fig. 18. That column, the ledger beam extending from the shear wall to it, and the double tees supported by the ledger beam and the planter box paralleling the double tees and providing a facing for the ramp's end had collapsed dropping the double tees and the planter box on to the floor below and also causing loss of several of its double tees. The planter box above the entry in Fig. 17 was also ripped off the building and dropped to the ground below.

At the roof level, the connections of the double tees to the east and central shear walls had all fractured. Also, the connector bars extending from the shear walls into the boundary elements connecting the ends of the double tees had failed. As shown in Fig. 19, the topping bars crossing the longitudinal joints between the double tees had also either fractured or buckled so that there was a complete failure of the top diaphragm from its east to west edge.

Shown in Fig. 20 is part of the top of Garage 8. This extensive structure used four-legged precast concrete "Conrad Tables" as the basic structural unit. Those tables were stacked up to three floors in height and a cast-in-place one-way post-tensioned slab spanned between them. At the ground level, the tables were supported on pedestals many of which had spalled badly, as can be seen in Fig. 21. This allowed the tables to settle in places by up to 8 in. (203 mm). Such settlement is visible adjacent to the column on the left in Fig. 20.

Lateral resistance was provided partly by shear walls and mostly by post-tensioning tendons that extended down through the legs of some of the

Fig. 31. Great Western Headquarters, Chatsworth.

Fig. 32. Cladding at Reliance Center, Glendale.

tables and were anchored in the pedestals. The concrete and the confinement reinforcement in the pedestals was inferior to that in the table legs so that the legs ground the pedestals away.

A fourth garage (Garage 25) in Glendale located across the street from the Galleria was visited. That five-level precast building is shown in Fig. 22. There was no observable damage to that building and it was in use at the time of inspection. According to local precast concrete manufacturers, there were an additional five garages utilizing precast concrete that suffered no damage.

Sherman Oaks Garages — 7 Miles (11 km) From Epicenter

Observations were made at five garages (Garages 9, 10, 11, 12 and 29) in the Sherman Oaks area. Only one of those garages (Garage 29), had a precast concrete gravity load framing system. It was three stories in height and constructed with double tees and precast concrete columns. It suffered only cosmetic damage to the exterior stucco. A second garage (Garage 12) had a simple steel frame with composite steel and concrete metal deck floors as the gravity load system, and masonry walls as the lateral load system. Those walls were badly cracked but repairable and the garage was able to function.

The remaining three garages used similar cast-in-place structural systems although their proportions were very different. That system comprised two shear walls in each direction to take the lateral forces. One-way concrete frames with post-tensioned beams supported one-way slabs post-tensioned transverse to the beams to take the gravity loads. It appeared that the earthquake triggered relatively large vibrational responses in those structures.

Shown in Fig. 23 is the east face of Garage 11. This garage was tall and relatively narrow. Inadequate separation of the spandrel panels from the columns had resulted in pounding between them, causing cosmetic damage to both the column and the panel. However, the vibrational effects had also been large enough to weaken the connection between the panels and the floor. In the upper right of the building, it can be seen that a panel is lost and in many bays wood bracing is providing a frame to which the panel could be attached while repairs were made.

Shown in Figs. 24 and 25 are details of Garage 9. In contrast to Garage 11, this building was a large one-level structure. It consisted of two almost identical structures extending in the east-west direction and separated by a seismic joint. Fig. 24 shows the south face of the east structure. The north-south movements of the garage caused spalling of the concrete especially at the back face of the exterior beam-to-column connection. The resulting debris are visible on the ground. The east face of the garage is shown in Fig. 25. There was spalling of the concrete

Fig. 33. Roof failure at Oviatt Library, CSU-Northridge.

along the length of its connection with the shear wall and buckling of the boundary element bars of the wall and the vertical bars of the central column. However, the garage was usable and was yellow tagged.

Shown in Figs. 26 and 27 are failures in Garage 10. This garage was of very similar construction to Garage 10, but with two supported floors and larger in plan. The building consisted of three separate structures with the western structure supported on piles and the central and eastern structures supported on spread footings. Fig. 26 shows the fractured exterior columns in the northwest corner of the building. Those fractures extended the length of the north face of the pile supported structure. At the interior wall to frame connection of that structure, the joint had fractured and the vertical bars had buckled more extensively, but in the same manner, as that shown in Fig. 25.

Shown in Fig. 27 is the appearance of a typical beam to interior wall connection for the area where the wall was on spread footings. The uplift in the asphalt shows that the spread footings of the wall have rotated and the spalling of the beam shows that gravity load elements resisted considerable lateral load. The concrete spalling has exposed the prestressing tendons.

The concrete frames with post-tensioned beams in all three of the foregoing garages were not intended to carry lateral forces. They obviously behaved otherwise.

West Los Angeles Garages — 14 Miles (23 km) From Epicenter

Two garages (Garages 13 and 14) in this area were visited. Garage 13 had apparently collapsed inwards in a manner similar to Garage 5 at CSU-Northridge. Some details of the construction of Garages 5 and 13 were apparently similar. For the north-south direction, lateral forces were taken by shear walls. In the east-west direction, lateral forces were taken by special moment resisting exterior space frames. Those frames paralleled interior gravity load frames extending in the same direction and were interconnected by an unbonded post-tensioned slab.

Like the CSU structure, this garage was only recently constructed. Removal of the debris was well underway when the investigators arrived and no useful observations could be made. A view of the collapsed structure, looking west in a direction essentially parallel to the frames, is shown in Fig. 28.

The second, older and larger, Garage 14 at this hospital complex was also closed. The damage did not appear to be extensive. On the long faces of the structure there was damage, as apparent from Fig. 29a, at the position where the precast column elements had been connected together, cracking of the north-south beams over their depth immediately above the face of the column haunch and damage to some of the spandrels due to pounding between them and the columns. More serious was the damage at the east shear wall (see Fig. 29b), where there had been uplift of the spread footing, tilting of the stairwell out from the building and significant damage of the connecting members between the stairwell and the building.

Santa Monica Garages — 14 Miles (23 km) From Epicenter

There were three garages (Garages 17, 18 and 19) in the St. John's Hospital complex, aligned in a north-south direction, connected to one another, and framing in at their south end to a wing of the hospital. That wing and the closest garage were separated by a seismic joint.

As shown in Table 1, the three garages had very different structural systems. The system that performed the worst was the central braced steel frame garage. Its buckled bracing and fractured columns are shown looking north in Fig. 30. The system that performed the best was the oldest, the reinforced concrete garage connecting to the hospital. The shear walls of the northern most garage had either developed severe diagonal cracking or fractured along construction joints.

Woodland Hills Garages — 6 Miles (10 km) West of Epicenter

Two garages (Garages 15 and 16) were examined in this area. Both were recently constructed and used the same gravity load structural systems as those of Garages 9, 10 and 11, and other structures. In Garage 15, where the gravity load frames with their post-tensioned beams paralleled the ductile moment resistant frames in the north-south direction, cracking could be seen in the column below the exterior beam-to-column joint. That cracking was clearly a precursor to failures of the type shown in Fig. 26.

Fig. 34. Wood frame of Devonshire Apartments, Northridge.

Fig. 35. First floor support system, Devonshire Apartments.

PRECAST CONCRETE CLADDING

Precast concrete is used widely for cladding in Southern California. Numerous observations were made of the performance of that cladding throughout the earthquake affected area. Very few cases of cladding damage to precast panels were found. Usually, that damage was caused by pounding between the cladding and an adjacent structural element due to inadequate separation. No cases of cladding loss, due to failure of the connections to the structural frame, were observed.

Shown in Fig. 31 is the Great Western Headquarters in Chatsworth. That ten-story office building used precast cladding and was only about 1 mile (1.6 km) from the epicenter. No cladding damage was apparent. Shown in Fig. 32 is the Reliance Center in Glendale. That 15-story office center was clad with stone-faced, precast concrete panels and located about 18 miles (29 km) from the epicenter in an area where several garages were damaged. There was no damage to the cladding of the Center.

As apparent from Fig. 17, Garage 7 had relatively heavy planter boxes attached to its face. Neither those boxes nor their attachments to the garage, showed any signs of damage. Garage 15 was faced with precast concrete panels. The connections of those panels to the exterior frame of the garage were closely examined and no cracking was observed. Those panels had been attached in accordance with PCI's recommended procedures.

Shown in Fig. 33 is the western end of the north face of the Oviatt Library at California State University-Northridge. The building was located about a half-mile to the west of Garage 5. This building had an addition in 1989 and precast cladding was added. There was considerable damage to both the north and south faces of the building at the roof line at the location where the facade stepped back on the south face. Dropping built-up steel roof sections caused spalling damage in the cladding and exterior columns.

Movements of the steel framed structure had also caused cracking in the base of some of the exterior precast columns. The building was closed. It has since been reported that upon close inspection, the joints in the steel columns where they attached to the foundation base plates were extensively cracked. Widespread cracking of steel joints was also discovered. Structural engineers have been surprised to find damage in several steel frame structures less than 5 years old.[6] Crucial welds were fractured and columns cracked or broke.

RESIDENTIAL HOUSING

Precast concrete components, frequently used for the first floor level and basement of condominiums and apartments throughout the Los Angeles area, performed uniformly well. These components allow secured parking below the living areas. Typical of that form of construction was the three-level wood frame Devonshire Apartment building located in Northridge about 3 miles (5 km) from the epicenter. The above grade portion of the building is shown in Fig. 34.

The basement and first floor level, constructed with cast-in-place concrete columns, precast concrete beams and hollow-core slabs, is shown in Fig. 35. No damage was observed in the precast concrete. However, repairs were being made to the above grade wooden portions of the building. This was one of seven buildings that were allowed to be occupied in a group of about 16 buildings. Many of the surrounding apartments were severely damaged and closed.

CONCLUSIONS

Based on the authors' observations at the site of the Northridge earthquake, the following conclusions are drawn. Some of these conclusions may need to be modified as more data become available.

1. Engineered structures, including those with precast concrete components, generally performed well throughout the region of strong ground motions. Exceptions were mainly buildings in the immediate vicinity of the epicenter, or those involving details known to be earthquake hazards.

2. Parking garages, particularly those with large plan areas, did not perform as well as other types of buildings. However, damage was not confined to any particular type of structural system. Partial collapses occurred in garages built with steel components and cast-in-place post-tensioned beams and slabs, as well as in those built primarily with precast components, or hybrid systems involving mixtures of systems.

3. In most damaged garages, the vertical elements of the seismic resisting systems, such as shear walls or appropriately reinforced columns, performed well. The greatest damage was in collector elements funneling loads to those vertical elements and in the columns of the structural systems that carried gravity loads and were not intended to be part of the seismic resisting system. In the latter case, damage occurred first in exterior columns, especially those in the same plane as a shear wall, or in interior columns loaded biaxially as a result of supporting transfer girders in two directions at right angles to each other.

4. No damage was observed in precast concrete cladding due either to inadequacies of those components, or inadequacies of their connections to the building's structural system.

5. No damage was observed in precast concrete components used for the first floor or first floor support of residential housing.

6. The damage observed in garages suggests that more consideration needs to be given by both designers and code rule committees to the factors governing:

(a) The extent to which the gravity load resisting elements can be considered to take in the deformations of the lateral load resisting system, deforming adequately while still not picking up lateral load, but providing vertical support to horizontal elements;

(b) The distribution of lateral load resisting vertical elements throughout the structure;

(c) The transfer of horizontal forces by collector elements to lateral load resisting vertical elements;

(d) The development of accidental torsions in shear wall buildings from features such as access ramps; and

(e) The factors governing ductile behavior of gravity load elements in the event of vertical overloads.

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REFERENCES

1. Department of Conservation, Division of Mines and Geology, Strong Motion Instrumentation Program, "Fifth Quick Report on CSMIP Strong-Motion Data from the Northridge/San Fernando Valley Earthquake of January 17, 1994," 8 pp.
2. Porcella, R., et al., "The M_s = 6.6 Northridge, California Earthquake of January 17, 1994: Selected USGS Accelerograms Recorded at National Strong Motion Network Stations," U.S. Geological Survey, 28 pp.
3. *Uniform Building Code,* International Conference of Building Officials, Whittier, CA, 1991.
4. NEHRP, Recommended Provisions for the Development of Seismic Regulations for New Buildings, Building Seismic Safety Council, Part 1, Provisions, 199 pp., Part 2, Commentary 237 pp., Washington, D.C., January 1992.
5. ATC-20, "Procedures for Postearthquake Safety Evaluation of Buildings," Applied Technology Council, Redwood City, CA, 1989.
6. "Steel Frame Fractures Raise Doubts," *Engineering News-Record,* March 14, 1994, pp. 8-9.