11 WCEE

Published by Elsevier Science Ltd 1996
Paper No. 650. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

CASE STUDIES OF THE PERFORMANCE OF RETROFITTED BUILDINGS DURING THE 1994 NORTHRIDGE EARTHQUAKE

by

MARTIN W. JOHNSON AND HENRI KOZA
EQE International, Inc.
18101 Von Karman Avenue, Suite 400
Irvine, California 92715

ABSTRACT

The 1994 Northridge Earthquake provided a strong test to several buildings in the San Fernando Valley of California for which seismic strengthening measures had previously been implemented. This paper presents three cases of seismically upgraded facilities and describes their performance during the earthquake. The performance was for the most part as planned for and as expected. The retrofitted buildings serve as testimonials to the fact that seismic retrofit of existing facilities can be feasible, cost-effective and can certainly reduce the risk to life safety and minimize damage to property if correctly implemented.

EIGHT-STORY INDUSTRIAL CONCRETE SHEAR WALL BUILDING

General Building Description

This building is located in Van Nuys, California, approximately three miles from the epicenter of the January 17, 1994 Northridge Earthquake. The published Modified Mercalli ground shaking Intensity (MMI) at the site was VIII. The building is a key component of a large industrial facility. It is an eight-story, slender, cast-in-place reinforced concrete structure. At the first and second levels, floor dimensions are approximately 90 x 100 feet, as shown in Figure 1. Vertical setbacks are located at the south side of the fifth level and at the sixth level. These setbacks create vertical irregularities in the structural system but also reduce the mass of upper stories. The total height of the building is approximately 143 feet, as measured from grade to the top of the seventh floor penthouse.

The building was constructed in 1953 in accordance with the Los Angeles Building Code. A first floor plan with typical wall thicknesses and locations of installed retrofit elements is shown in Figure 1. Elevation views showing the building before and after retrofit construction are presented in Figures 2 and 3.

Floors and roofs consist of one-way concrete slab-on-beam construction. Floors are supported by the exterior and interior concrete walls and interior reinforced concrete columns. Most of the walls are 10 inch thick cast-in-place concrete with intermediate grade (40 ksi yield) deformed bar reinforcing. Wall reinforcement typically consists of #4 or #5 bars at 10 inches to 18 inches on center.

Foundations consist of drilled caissons extending to a depth of 50 feet below grade. The caissons have 24- to 30-inch shafts and bear with widened bells on a dense layer of sand. The caissons are interconnected through a series of reinforced concrete grade beams. The slab-on-grade is 6 inches thick with #3 reinforcing bars at 16-inch centers.

The building's lateral force-resisting system originally consisted of concrete floor and roof slabs which transmitted lateral forces to reinforced concrete shear walls at the interior and perimeter of the building. The tapered vertical geometry of the building is advantageous to the seismic performance of the building in that there is relatively less weight in the upper floors where the dynamic response is the greatest.

Description of Retrofit Installed Before the Northridge Earthquake

In a preliminary analysis of the building, several areas of concern were identified. The design forces defined by current building codes for the building were found to be up to 100% greater than the original forces for which the building was designed. Several of the shear walls in the building were found to be significantly overstressed. In addition, due to the vintage of design, walls were not designed to include boundary elements for overturning considerations. Wall ends typically did not contain any "localized" reinforcing to resist overturning forces from the higher, current building code forces.

Direct shear forces in many walls of the unretrofitted structure greatly exceeded the code prescribed shear capacity. For example, the main shear wall along Column Line C had wall shear demand/capacity (D/C) ratios of 2.01 to 2.36 from the sixth floor level down to the third floor level. At these floor levels, this wall is a main shear wall in the east-west lateral force-resisting system. Below the third floor level, the lateral force-resisting system contains additional shear walls due to the nature in which the structure expands into a larger footprint.

In the north-south direction, the perimeter wall along Column Line 2 is a primary element of the lateral force-resisting system. In the unretrofitted structure, this wall was found to have wall shear D/C ratios that ranged from 1.19 to 1.94 over the height of the wall from the fifth to the second floor levels.

As a result of this analysis, the building was rated a high seismic risk. While total collapse was not thought likely, damage to walls and other lateral force-resisting elements may have led to extensive localized wall cracking and potential local floor collapse.

In support of this analysis is the fact that this structure experienced earthquake damage from the 1971 San Fernando earthquake. Accelerometers about 1/4 mile from the site registered ground motions of about 0.25g in the north-south direction, and about 0.13g in the east-west direction (these levels are about 1/2 to 1/3 the levels experienced in the January 17, 1994 Northridge Earthquake). Damage reports are sketchy from that earthquake, however, extensive cracking in walls was reported. It was also reported that large tanks on the upper floors of the building were temporarily empty at the time the earthquake struck, thus minimizing the amount of inertial load generated in that earthquake.

After the initial building analysis was conducted, the building was retrofitted by EQE in 1989 to a level of seismic strength and safety intended by current codes, and within the owner's objective for operability following a significant seismic event. Figures 2 and 3 show the subject building before and after the structural seismic retrofit. Note that openings were infilled in the upper floors, and while not apparent from the photograph, walls at the lowest level in line with the infilled walls above were also thickened.

The general approach to strengthening the building was to strengthen the structure in each of the principal directions to reduce story drift to a level compatible with existing structure elements. The building was analyzed and retrofitted to a base shear of 0.25g. This base shear is not only twice as high as the original design base shear, it is also higher than the minimum base shear required by the Uniform Building Code (UBC) at the time of retrofit design. The reason for this is that the client wanted a level of facility functionality immediately following the earthquake.

In the east-west direction, Column Line C (see Figure 1) was chosen for strengthening because of the suitability of this wall to provide significant resistance to seismic forces. The shear wall at this line is solid, continuous throughout the height of the building, and located in the central region of the structure.

Strengthening measures in the north-south direction were incorporated along Column Lines 2 and 6 (see Figure 1). The building did not have an interior north-south column line suitable for strengthening. Both Column Lines 4 and 5 are generally discontinuous through the height of the structure, and neither had enough existing "solid" wall surface to make strengthening along these lines feasible. Strengthening lines 2 and 6 essentially provided for "solid" shear walls from the upper levels to the foundation, and also resulted in a more symmetric layout of earthquake force resistance.

New foundations, in the form of reinforced concrete grade beams, were added under the new and thickened shear walls. The new foundations were doweled into existing foundations using adhesive anchors. Existing caissons were deemed adequate.

Other typical building retrofit details included dowels between the new and existing walls, and new grade beams doweled into the existing foundation. Boundary elements were strengthened by adding additional reinforcing bars at the ends of new walls, or by adding high strength steel plates to existing columns which became an integral part of the new shear walls.

The retrofit did not attempt to reduce the overstresses in every element of the building nor did it attempt to provide full ductility up to the standards of the current building code. Analysis of the retrofit implemented retrofit scheme indicated that at the 0.25g base shear level, some shear walls in the building remain overstressed even after strengthening. The overstresses were taken as an indication of potential local damage but overall the strengthened structure was expected to perform at the desired level. The primary objective of the strengthening measures was to reduce wall and boundary element overstresses throughout the structure as compared to the unstrengthened structure. The infilled openings and thickened walls provided additional seismic resistance and redundancy, both positive factors for improved seismic performance.

Because not every overstressed element was strengthened, the expected performance of the structure was checked by conducting analysis with computer structural models by reducing the stiffness of overstressed elements (particularly minor shear walls, but also coupling beams) to reflect their damage and resulting degradation. These studies indicated that earthquake forces were re-distributed to the strengthened elements, and that the structure as a whole would perform as desired.

Retrofit construction took approximately 8 months, and no disruption to ongoing facility operations occurred. The only off-shift work occurred with the installation of dowels and formwork for concrete infill of an existing window opening located at a room with spontaneous blast potential. This work was scheduled and completed during a normal, short duration plant shut-down.

Observed Earthquake Damage

Nearby recorded ground motions during the Northridge earthquake showed a peak ground acceleration of about 0.3g. This resulted in spectral accelerations to this building of about 0.6g, at both the structure's fundamental north-south and east-west periods of approximately 0.4 seconds.

No major structural earthquake damage was observed immediately after the main event of January 17, 1994 and the building was tagged "green" by local building officials.

Following the subsequent aftershocks, minor cracks were observed in shear walls at lines C and F at the first story and lines A and 6 in the upper stories. Cracks were observed primarily above door openings and in taller, slender walls. The cracks were all considered minor to moderate and were repaired by epoxy injection. Many of the cracks were suspected to be a re-opening of cracks improperly repaired after the 1971 San Fernando Earthquake. No downtime was recorded, and operations were able to resume immediately after the earthquake.

Summary and Conclusions

The building, which had been retrofitted prior to the earthquake for a level of force greater than that specified by the UBC, was operational immediately following the earthquake. The building's performance level can be characterized as "Immediate Occupancy" per the 50% draft of ATC -33. The walls, foundations and boundary elements remained essentially elastic during the Northridge Earthquake. A combination of lateral strength capacity, element geometry, and well defined lateral force paths combined to provide the desired building performance.

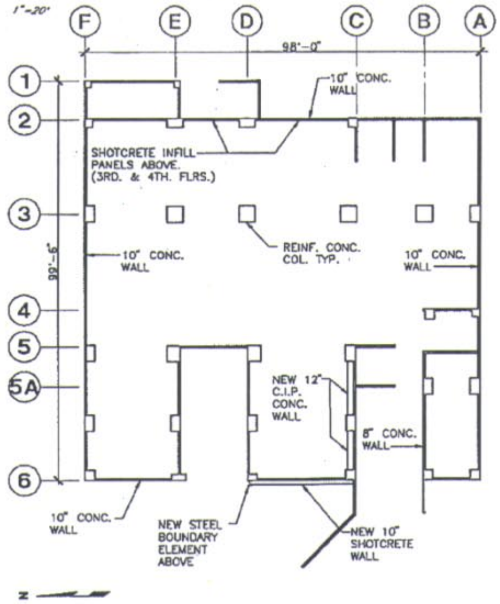

Figure 1: Industrial Building In Van Nuys - First Floor Plan
(Retrofitted Areas Highlighted)

Figure 2: Elevation of the Industrial Building In Van Nuys Before Retrofit

Figure 3: Elevation of the Industrial Building In Van Nuys After Retrofit

THREE-STORY COMMERCIAL RETAIL BUILDING

General Building Description

This building is located on Victory Boulevard, in Canoga Park, California about 4 miles from the epicenter of the January 17, 1994 Northridge Earthquake. It is a three-story, concrete shear wall structure, with a footprint of approximately 135 feet by 270 feet in plan, and with a total height of about 54 feet. The "lower mall" level is the basement of the building; the "upper mall" is the grade level; and the "fashion level" refers to the upper or third level of the structure.

The original building was constructed circa 1964. Both roof and floor systems consist of relatively heavy concrete waffle-slab systems which are supported by perimeter concrete bearing walls and by 22- and 24-inch square interior concrete columns. Building foundations consist of concrete pile caps, grade beams, and tapered, mandrel-driven piles, each about 35 feet long. Site soils consist mainly of alluvial deposits (sands and silts) with a relatively shallow water table.

Most of the walls are 10 inch thick cast-in-place 3,000 psi minimum concrete with intermediate grade (40 ksi yield) deformed bar reinforcing. Wall reinforcement typically consists of #4 bars at 18 inches max. on center each way.

No evidence of any significant past structural damage resulting from previous earthquakes near the area (e.g., the 1971 San Fernando Earthquake) was observed or referred to during site visits.

Description of Retrofit Installed Before the Northridge Earthquake

In 1988, a detailed evaluation was conducted which indicated a potential for poor earthquake performance due to a weak story located at the "upper mall" (or middle floor) level of the building. To mitigate this, earthquake retrofit measures consisting primarily of shotcrete-infilled shear walls were constructed in 1989. These retrofit wall locations are shown in Figure 4.

The 1989 retrofit design was performed using the equivalent lateral force method, scaled to 100% of the force defined by the 1988 Uniform Building Code (UBC). However, selected existing elements were permitted overstresses of as much as 25% if the basic "life safety" design objective was maintained. Thus, the evaluation of existing elements essentially mirrored the provisions of ATC-14. Where stresses exceeded this 25% overstress level, sufficient new strengthening elements were added so that both the new and the affected existing elements would meet the minimum-strength criteria of the UBC.

Retrofit construction consisted of 4,000 psi minimum concrete for 10-inch thick shotcrete walls reinforced with #4 bars ($F_y = 60$ ksi min.) at 18 inches maximum spacing each way.

The new walls were attached to the existing concrete by reinforcing bars which were doweled with epoxy for a short distance into the existing concrete. The length of embedment of the epoxied dowels into the original concrete was sufficient to transfer the calculated seismic forces, but was short of the length required to develop the full tensile capacity of the reinforcing bars. This short embedment was not a direct factor which was associated with any of the damage observed following the Northridge Earthquake, but may have been a secondary factor where localized crushing and spalling of the surrounding concrete allowed portions of the dowels to become exposed.

Observed Earthquake Damage

Nearby recorded ground motions during the Northridge Earthquake showed a peak ground acceleration of about 0.5g. This resulted in spectral accelerations to the building of about 0.8g and 0.7g, corresponding with the structure's fundamental north-south and east-west approximate periods of 0.2 and 0.1 seconds, respectively.

Following the Northridge Earthquake, the building was initially "red-tagged" by local building officials due to observed structural damage. This was later revised to a "yellow tag," after a letter summarizing initial evaluation findings was provided by engineers to the building department. Figure 4 depicts typical general damage observed at an exterior elevation. Temporary shoring posts utilizing steel columns were installed in several areas immediately after the earthquake to stabilize the building.

Structural damage consisted of severe damage to the retrofit shear walls between column lines G and H, as well as moderate crushing and cracking damage to the original-construction shear walls between column lines D and F, at both the east and west sides of the building. Less noticeable, but equally significant, was approximately 2 inches of sliding damage that occurred at the base of each of the original north-south shear walls, along the construction joints between the walls and their foundations. Vertical reinforcing bars at these joints are equal in size and spacing to the wall reinforcing and 100% of the bars are spliced across the joint with a lap length of 12 inches. This compares with current code lap splice requirements of about 30 diameters, plus 50% staggered splices.

Overall, the construction joint detail was marginal to transfer the wall shear forces, if it is assumed that the construction joints were well-roughened and cleaned before the wall was cast. However, although damaged reinforcing bars were observed in the earthquake-damaged joint areas, the damage observed was typically bending and breakage that was associated with the large lateral displacements. No signs of rebar withdrawal or splitting were observed that might have been associated with a lap splice or development failure. A possible explanation for the sliding failure may therefore be that some combination of insufficient roughness, inadequate cleanliness, and poor concrete bonding may have created a reduced shear-friction capacity along the base of the wall.

Other types of damage that occurred during the earthquake included structural damage to a mechanical penthouse and roof screen, damaged plaster ceilings, and extensive flooding damage caused by broken water pipes. Overall, the pattern of damage indicated a pattern of predominantly north-south shaking.

Because the wall thickness and reinforcing quantities of both the original and retrofit walls were equal, while the minimum specified strength of the concrete and reinforcing steel used in the retrofit wall was greater than that used in the original construction, it was initially surprising that significantly greater damage was observed in the retrofit construction than in adjacent original-construction walls. However, the discovery of sliding shear at the bases of the original-construction walls suggests that north-south seismic forces were almost entirely resisted by the retrofit walls adjacent to the sliding walls.

Also interesting is the fact that cracking which occurred in the retrofit shotcrete walls tended to occur along horizontal planes, rather than as diagonal cracks. Horizontal shearing and sliding mechanisms normally only occur in concrete walls along construction joints, where a weakened plane is present in the concrete. A similar weakened plane in the building's shotcrete walls might have initiated during construction due to shrinkage cracking in the shotcrete wall, if the shotcrete bonded to the existing concrete at the perimeter and shrank away from the center, or if the sliding occurred along the construction joint created between adjacent "lifts" of shotcrete. This second possibility is considered more likely.

Unlike normal concrete walls, retrofit infill walls do not support significant vertical loads (because the concrete is placed after all building loads are already in place). Therefore, little weight is present to provide friction sliding resistance once an initial horizontal plane of sliding is formed within the wall. Also, the aggregate size used in shotcrete construction tends to be relatively smaller than that used in cast-in-place concrete construction. These two causes may have led to considerable hysteresis "pinching" and stiffness degradation in the retrofit wall. This would also tend to reduce the post-elastic damping available.

Overall, the building performed largely as intended by the retrofit design - collapse was avoided and the building was completely restored to use within a reasonable timeframe (6 months).

Summary and Conclusions

Overall structure performance was within the scope of code definition of performance for ordinary structures, and it appears that the presence of the retrofit construction provided a significant degree of resistance and energy dissipation that acted to reduce greater damage that would have otherwise occurred in the structure. However, the inadequate attachment of the original-construction walls to their foundations reduced the total resistance provided and led to greater damage than would otherwise have occurred. Also, shotcrete-constructed structure elements should have given better performance in terms of the mechanism and pattern of failure. The actual performance might have resulted from inherent differences of the infilled-shotcrete construction process from that of conventional cast-in-place concrete construction.

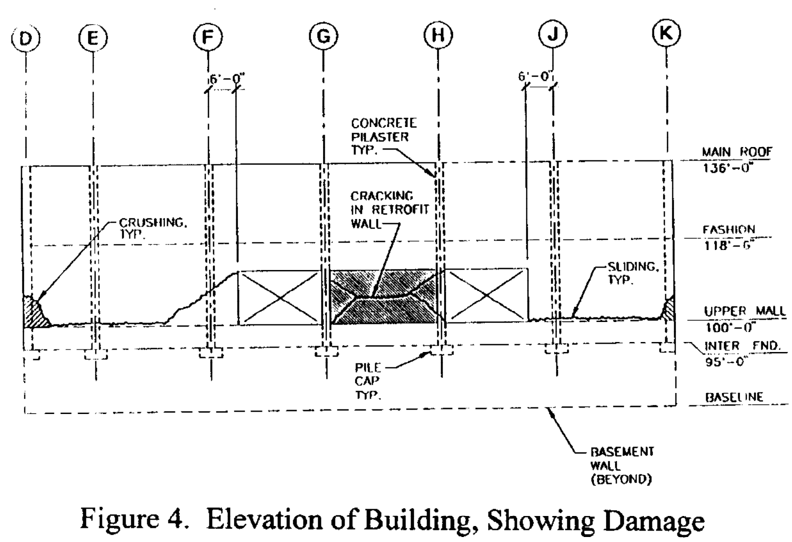

Figure 4. Elevation of Building, Showing Damage

TWO-STORY NON-DUCTILE CONCRETE FRAME OFFICE BUILDING

General Building Description

This building is located on West Valencia Boulevard, in Santa Clarita, California. It is a two-story, concrete frame structure, with a footprint of approximately 132 feet by 252 feet in plan, with a total height of 24 feet. The building has a long, narrow skylight/atrium opening penetrating the long axis of both the roof and second floor levels. A photograph of the building exterior is shown in Figure 5.

The office building is located immediately across the street from the Santa Clarita City Hall, which suffered extensive damage and racking during the Northridge Earthquake due to fractures of welded steel moment connections.

The roof and second floor systems of the building consist of post-tensioned concrete flat slab systems supported by 30-inch square concrete columns. There were no drop beams spanning between columns or thickened pan or capital areas in the vicinity of the columns. Each of the columns is supported by 2-foot thick reinforced concrete pile caps and three 24-inch diameter drilled concrete piles, each about 50 feet long.

The original design of the building was based upon the 1975 edition of the *L. A. County Building Code*, which is roughly equivalent to the 1973 UBC. Before strengthening, lateral forces were resisted entirely by moment-frame action between the slabs and the columns and by fixed-base bending of the columns against the pile cap foundations.

Flat-slab buildings which rely on moment-frame behavior to resist lateral forces can be high seismic-risk structures, primarily due to bending moments and punching shears which can occur at slab/column joints. After a detailed evaluation confirmed a potential for poor earthquake performance, earthquake retrofit measures were implemented in late 1990.

Description of Retrofit Installed Before the Northridge Earthquake

Earthquake strengthening measures added new concrete beams at select locations on the top surfaces of both the floor and roof levels. These beams were sized and reinforced to cause flexural yielding of the new beams rather than in the existing columns or in the new beam-column joint of the existing columns.

Because the owner requested "post-earthquake operability" in addition to basic life safety, the retrofit design was performed using a site-specific response spectrum, scaled to 100% of the force defined by the 1988 Uniform Building Code (UBC), and including an Importance Factor, I = 1.25.

"Ductile moment frame" SMRF criteria was used as much as possible or practical to design the retrofit. However, because some elements of ductile detailing could not be adhered to in the retrofit design, an Rw factor of 7 was used for the design, which is equal to the basic UBC "intermediate moment frame" value.

Figure 6 shows the typical beam and beam-to-column joint reinforcing at the second level (similar for the roof level), and Figure 7 shows the finished beams at the second level, which blend well with the existing architectural features of the building.

Observed Earthquake Damage

The building is located approximately 13.5 miles from the epicenter of the Northridge Earthquake. Site soils consist mainly of alluvial deposits (sands and silts). Due to the moderately soft soil conditions, with groundwater frequently within 30 feet of the ground surface, there is a potential for some liquefaction at the site. However, no signs of any liquefaction were observed near the site.

A CDMG station at the Newhall - L.A. County Fire Station, located about 2 miles from this building, recorded ground motions with very strong horizontal spectral accelerations of about 2.5g and 2.1g, corresponding with the structure's first and second mode periods of about 0.3 and 0.7 seconds, respectively. Another nearby spectrum recorded at S.C. Edison's Pardee Substation had a peak acceleration of about 1.5g at a period of about 1 second. Site soil conditions at both recording stations were similar and consisted of alluvium. The difference in response might have been due to a soil-resonance mode within the alluvial valley materials, in which deeper or less dense soils caused a portion of the site spectrum to "peak" locally.

Considerable office disarray occurred during the earthquake, in terms of overturned furniture, etc. However, no structural damage (including any visible cracking to concrete elements or broken glass) was discovered either immediately after or following the numerous aftershocks. The structure was therefore considered to have responded elastically to the earthquake motions.

Summary and Conclusions

This building, which had been seismically retrofitted to the design requirements for an "essential facility" in accordance with the 1988 UBC, performed very well during strong shaking experienced during the Northridge Earthquake.

Figure 5. View of Building Exterior

Figure 6. Beam and Joint Reinforcing at Second Level

Figure 7: Finished Beams at Second Floor