PART B. ENGINEERING ASPECTS ON THE JANUARY 17, 1994 NORTHRIDGE EARTHQUAKE

1. LIFELINE DAMAGE

Lifelines are complex systems which distribute resources, transport people and send information The water supply, water treatment and sewage, electric power supply, communications, transportation of people and of combustible materials (liquids and gases) are examples of lifelines. Although at present only some components of lifelines are explicitly designed to resist earthquakes, the engineering practice of such systems has been improving since lifeline behavior is starting to be considered important in seismic design, in emergency planning, and in the recovery after an earthquake. Current experience always shows that when a large earthquake occurs, there is considerably amount of lifeline damage.

Distinctly from buildings, lifelines are generally parts of networks which may extend over several kilometers. Therefore, they can pass through zones with different types of soils or rock. Lifelines are interdependent systems, which means that damage in one of them influences the behavior of the others. For example, damage or disruption of the electric power supply can affect the operation of water pumping, thus limiting the available amount of water; moreover, in order to repair the electric power system, a redundant transportation system is necessary.

During the Northridge earthquake, most lifelines showed adequate redundancy; therefore, large service interruptions were avoided. The transportation system was the exception.

1.1 Water Supply System

According to local government information, three days after the earthquake (January 20), 36 000 people suffered from shortage of water supply. By January 25, only 2,000 persons lack of water supply due to the work of 50 crews. Most of the pipeline fractures occurred near the epicentral zone

1.2 Gas Distribution System

Although there are not statistics on damages, it has been estimated that damages in the gas distribution system will be less than those in the water supply network. One day after the earthquake, about 15,000 to 20,000 customers lost gas supply. By Friday 21, the number of affected customers increased to 40,000. One of the most important fires after the earthquake occurred in Granada Hills, a residential zone north to the epicenter. Here, a gas leak from a 60–cm diameter main parallel to a water pipe, caused the explosion. Several wooden houses were burnt mainly because of water shortage for extinguishing the fire.

1.3 Electric Power System

After the earthquake, 150,000 customers had no service; but by January 21 only 2,300 customers remained without electricity. Damage was concentrated in one 500kV substation and in two 230kV substations from the Sylmar Power Station (Fig. 22), NW to the epicenter. This power station was also damaged in the 1971 San Fernando earthquake. At that time, this damage showed the need to fix the electrical equipment with adequate anchorages. In fact, at that time, because of anchorage deficiencies, several transformers and other components were turned over and failed causing severe damage. As a result, components and equipment were replaced by new ones designed to resist a 0 5g acceleration level and also fixed with adequate anchorages. Nevertheless, during the Northridge earthquake, instruments in one of the substations registered ground accelerations of 0.6g. Several electric components were again severely damaged

The ceramic components are one of the most vulnerable parts to earthquakes actions, particularly in 220kV equipments or bigger. While the development of high-strength ceramic or of isolators with a more ductile materials is done, isolation systems must be designed for low stress levels. Other option is to replace the cantilever supports for the isolators with multiple supports. The use of dampers has shown to be adequate.

1.4 Communication System

Moderate damage was suffered by the equipment of telephone central stations. Some of these centrals went out of service due to the interruption of the electric power supply and the failure of emergency plants Some phone circuits were overloaded after the earthquake. The low level of damage contrasts with that observed after the 1971 San Fernando earthquake. At that time, some switch boxes overturned since they were not laterally braced. Failures observed in 1971 forced telephone companies to develop minimum requirements for buildings and bracing systems.

1.5 Transportation System

During the 1971 San Fernando earthquake, the road and bridge system near Los Angeles was considerably damaged. Five freeway bridges collapsed and 42 more had various different levels of damage. In the aftermath of this experience, a complete revision of the seismic design criteria for bridges was carried out. According to the old criteria, a base shear coefficient equal to 0.06 was assumed. This value had been used since 1943. After the revision, published in 1974, seismic forces were obtained as a function of the maximum possible acceleration on rock, of the soil profile, and of the ductility level, and risk of the structural system. Base shear coefficient was increased approximately to 0.1. Since that time, design standards for bridges have not been considerably improved Because of the damage reported in 1971, the California Transportation Department has incorporated the use of dynamic analysis and of results from research for designing bridges

Almost the entire 1,000 km of freeways in Los Angeles area resisted the 1994 earthquake without considerable damages. As a reference, three million cars move everyday in this area In most cases, rehabilitated structures (steel jacketing of columns (Fig 23)) by Caltrans since the Loma Prieta earthquake (1989) behaved well (Fig. 24) Nevertheless, damage in 10 bridges was observed (Figs. 25 to 27). Main damage occurred in Interstate Highway 5 (at the junction with Roads 118 and 210), in Interstate Highway 405 and I-10; and in Roads 101 and 118. Three types of failures were observed a) Column failure due to shear-compression (Figs. 26 and 27); b) Column flexural hinging (Fig. 28), and c) Beams fell down because they lost their seats.

The junction of the I-5 with R-14 was under construction when the San Fernando earthquake occurred; at that time, the structure partially collapsed and showed damage in other parts. This same junction collapsed in the Northridge earthquake, possibly due to shear failure of columns. These columns have a small height to depth ratio. This feature increased the stiffness and, consequently, augmented the magnitude of the lateral force in this element. Other junction which suffered damage in both earthquakes, 1971 and 1994, was that of R-5 with R-210.

The most common failure mode was brittle shear or semiductile shear (flexural hinging first and shear failure after). For example, in I-10 near La Cienega, concrete columns showed an inclined failure plane and concrete crushing in the zone where longitudinal reinforcement buckled (Fig. 29). Possibly, high horizontal accelerations caused the column to fail in shear while the vertical acceleration added to the deterioration. According to Caltrans, the I-10 bridges had been identified as highly vulnerable structures and there were to be rehabilitated soon

Some bridges, which had been rehabilitated with restrainers (cables) to limit excessive longitudinal displacements, collapsed after the beams lost their seats.

Other type of failure was pounding of structures, especially in skewed bridges, where in–plan column distribution generated important torsional effects. In-plan rotations produced large displacements and forces at the end of members sometimes causing the fracture of restrainers used to limit displacements. In some cases, site effects may have increased the ground motion, that is the case of I-10 (located in La Cienega, at 23 kilometers from the epicenter), founded on soft soil. Topographic conditions of this canyon, where the R-5 and R-15 junction is located, might have affected the ground motion.

2. HOSPITAL DAMAGE

Among the emergency group of buildings and installations are hospitals, police and fire stations, civil protection units, rescue units, etc. Obviously, hospitals are buildings which must remain without significant damage to attend injured victims. Modern building codes, like the Mexico City Building Code (RDF87), consider a larger safety factor for this type of structure.

Thus, in RDF-87, design forces are increased by 50%, while in the actual Uniform Building Code adopted in the State of California, design forces are increased by 25%.

In the case of Northridge earthquake, nine hospitals were damaged and were closed temporally. By January 21, only three hospitals remained closed.

One of the damaged hospitals was the Santa Monica Hospital which is a R/C building with concrete structural walls perpendicular to the street, and with frames (beams and columns) parallel to the street (Fig. 30). During the reconnaissance evaluation, the entrance to the building was restricted Damage was concentrated in the concrete walls and coupling beams (Figs. 31 and 32). Diagonal cracks were observed in R/C walls, which are evidence of high lateral forces. Horizontal cracks were also noted and agree with the location of construction joints. The latter type of cracking occurs along inadequately prepared joints between two castings. Indeed, construction joints must be carefully prepared removing the dust, slag or any other material which can impair bond between the old and new concrete. Construction joints must be roughened to increase the coefficient of friction; vertical rebars must pass through the joint to prevent and control possible cracking. In some cases, problems in construction joints can be explained by deficiencies in the detailing by the structural designer. Regarding coupling beams, tests carried out in laboratories and recent experiences during earthquakes have demonstrate that this type of beams can be detailed relatively easy (by placing diagonal reinforcement); beams designed accordingly have shown excellent response without severe damage. The coupling beams in this building were designed before the knowledge in this field was developed.

Other severely damaged structure was the St. John's Hospital located in Santa Monica. This is a 6-story R/C building with an appendix; possibly, the structure was built 40 years ago. Only the second story at the north facade suffered damage. Columns and short walls showed large diagonal cracks. Damaged was caused by a drastic change in strength and stiffness in this story with respect to others (Fig. 33)

The Olive View Medical Center (identified as Sylmar Hospital) is a 6-story building plus a basement (Fig. 34). It has a central core of R/C walls and a ductile moment-resisting space steel frame. The ground and first floors have a square plan while the remainder of the building has a cruciform plan. At the end of the cross arms there are structural walls made with steel plates. The foundation consists of R/C walls and slab. The structure is located in an alluvial plain, 15 kilometers away from the epicenter. The floor system was built using steel decks covered with concrete. Beam spans are 6.5 m and story height is 5.4 m. This hospital substitutes the hospital building that collapsed during the 1971 San Fernando earthquake. In this building, structural walls in the upper stories were interrupted at the ground story. This situation caused the collapse of exterior stairs and tilting of the building. Damage in ground floor columns, particularly in corner columns, showed the inadequate detailing of these elements with small diameter hoops with large spacing. The maximum estimated horizontal acceleration in the 1971 earthquake was 0.5g. The actual hospital, with 350 beds, was built in 1975, and it was not occupied until 1985, after satisfying the strict requirements for safety and hygiene of California. During the reconnaissance, the only structural damage observed were diagonal cracks in the basement walls (box foundation) which were 0.3 mm width and were spaced at almost 20 cm. Basement columns are R/C elements with a square transverse section with 50 cm sides. These columns were harmless.

Maximum recorded horizontal accelerations at the base of the building (inside the foundation) were 0.82g while in the roof level increased to 2.31g. In free field, maximum recorded horizontal and vertical accelerations were 0.91g and 0.60g, respectively. It is important to note that a high level of design base accelerations for hospitals is about 0.3g.

In the basement, office files with patient information overturned and some ceiling panels fell (Fig. 35). The upper most two stories were flooded due to a fracture of 2.5-cm diameter copper pipes. Also, seven fractures of the black iron pipe of the fire sprinkler system were identified. Although there was no structural damage, the water damage obligated the hospital evacuation.

At the roof of the building, horizontal measured accelerations during the earthquake were 2.31g. A lamp detachment was observed. An air extractor equipment slipped due to an inadequate detail in the anchorage supports (Fig. 36). Anchor bolts were cut just above the nut. Shaking caused by the earthquake caused ripping of the bolt's thread thus allowing displacement of the equipment. Similar incipient failures in other equipments with the same anchoring detail were observed.

Other damaged hospital was the Indian Hills Medical Center. This structure had suffered damage during San Fernando earthquake. At that time, four R/C structural walls showed many diagonal cracks. The structure was rehabilitated by epoxy resin injection and by increasing the wall thickness. During the Northridge earthquake the building showed damage in the structural walls, particularly crushing and failure of longitudinal steel overlaps at the fourth level.

3. DAMAGE IN BUILDINGS

As a result of this earthquake, only a small percentage from the total economic loss is attributed to structural damage. Damage in non-structural elements and installations led to important direct and indirect economic losses. The latter is due to disruption of business operations. Building damage was distributed over a large area, although the most affected zones were Northridge, Canoga Park, Hollywood and Santa Monica. As it was mentioned before, in La Cienega District, in Santa Monica, it is probable that local soil conditions might have amplified the ground motion.

3.1 Dwellings

It is commonly accepted that wood is a good material to be used in seismic areas.

Most dwellings in the epicentral zone were one-story timber houses and timber apartment complexes In general, wood has a ductile behavior when compressed, while under tension (in particular in the orthogonal direction to the grain) its behavior is brittle. Against the belief of such good performance, earthquakes have shown the vulnerability of structures with different anchoring systems and with ground floors with large open spaces used as parking. It is interesting to mention that several structures with these features collapsed during the 1971 San Fernando and the 1989 Loma Prieta earthquakes. The Northridge event was not the exception.

The structural system in American housings is similar and is made of a basic timber frame which is used several times in a house. Loads and spans are small. Some buildings have relatively heavy unreinforced masonry chimneys; some structures are often covered, partially or totally, with a facade finishing. In a well designed and detailed structure, lateral loads are transferred from floor diaphragms to structural walls. Structural walls are walls covered with stucco or plywood (if they are exterior elements) or with a gypsum board (if they are interior walls).

Preliminary estimates give a toll of 15,000 damaged houses; most of them with non-structural distress In this number, the collapses of reinforced and unreinforced masonry chimneys (with a replacement cost which varies between 5,000 and 10,000 dollars) and roof damage are included. Masonry fences were also damaged; some collapses (overturning) were recorded.

A characteristic of the damage pattern produced by the Northridge earthquake was that observed in multi-story timber frame structures. Several of them were lacking of wood structural walls. The collapse of the Northridge Meadow Apartments complex caused 16 fatalities. The structural characteristics of such type of buildings are similar: they are two- or three- story wood buildings with a ground soft story used as parking (Fig. 37). These apartment buildings were designed and constructed prior to the 1975 recommendations which required structural walls to be made with plywood sheets nailed to the frame (Fig. 38) During the earthquake, large lateral displacements were concentrated in the ground soft story (Fig 39). In some instances, the structure was left standing but severely damaged; other buildings collapsed over the soft story. In other cases, failure was observed along the joints between the concrete masonry foundation (reinforced masonry) and the timber structure, and between upper wooden stories.

3.2 Masonry Buildings

Main causes of damage and collapse of masonry structures in past quakes in different parts of the world have been. 1) lack of anchorage of roof or floor beams to walls; 2) inadequate detailing of beam-to-wall anchorages; 3) out-of-plane wall flexural failure, 4) diagonal cracking of the masonry panels; 5) excessive distortion of floor diaphragms; and 6) deficiencies in the shear transfer mechanism between floor diaphragms and walls All these types of damage were observed in the area affected by the Northridge earthquake (Figs. 40 to 42).

In 1981, Los Angeles County started a rehabilitation program of unreinforced masonry buildings. Most of this type of structures, built at the beginning of the century, have flexible wooden floor system, generally not anchored to the walls. Multi-story buildings possess R/C slabs supported on walls. Rehabilitation techniques have included stiffening of roof and floors and bolting them to walls (Fig. 43). Most rehabilitated structures withstood the 1994 event, although in few cases severe damage was recorded. Buildings not retrofitted experienced large diagonal cracking, out-of-plane collapses and considerable brick crushing.

Based on this experience, it is clear that the rehabilitation program has not been completely effective; however, it did contribute to reduce this type of structure's vulnerability

3.3 Reinforced Concrete Buildings

A separate section is devoted to parking structures due to their outstanding damage rate.

Observed failure modes in R/C structures included: 1) diagonal cracking and concrete crushing in poorly detailed beam-column joints; 2) shear failure at beam and column ends with improper detailing; 3) bond and anchorage failures of steel reinforcement (Figs. 44 and 45); 4) shear failure of coupling beams (Fig. 46); and 5) shear failure (brittle) in short ("captive") columns with height-to-width aspect ratios of 2.5 or less (Figs. 47 and 48).

The instrumented building located closest to the epicenter (7 km) is a R/C frame built in the 60's and damaged in the 1971 event (Fig. 49). In the Northridge earthquake the structure was subjected to a base acceleration equal to 0.47 g; a 0.59 g maximum acceleration was recorded at the roof Maximum recorded vertical acceleration was 0.30 g which occurred before the horizontal maximum (this was the case for all records). Although this building is not greatly damaged, its demolition is probable.

Tilt-up construction, used for industrial buildings and warehouses, has behaved poorly during earthquakes. In such type of structures, floor diaphragms are made either of wood or metal. During the 1994 event, failures were located at the wall-roof connection. The collapse of one tilt-up building was reported, in this structure the roof bolts were ripped off from the walls.

It was reported that the Los Angeles Coliseum suffered considerable, but repairable, damage along a joint between two portions of the structure.

3.4 Parking Structures

Parking structures was the most damaged type of building designed according to present requirements and criteria. Most parking structures in the US have typical spans of 16 to 20 m, to give ample freedom for selecting the layout of car spaces. Since the design is often controlled by the spans to be bridged, precast prestressed elements or cast-in-place post-tensioned members are used. Parking structures located in seismic zones require laterally rigid structural systems to control displacement and, simultaneously, flexible systems to reduce possible distresses caused y volumetric changes of the concrete (shrinkage, creep, etc.).

For short-span parking structures, flat plate and flat slabs are the most typical system. For long spans, two systems are used. For cast-in-place structures, beams and one-way slabs, both post-tensioned, are employed. If precast elements are used, double-tee beams (60 to 80 cm deep) are common. A 5- to 10-cm thick concrete topping is cast on the beams as a riding surface; the topping is also intended to contribute to the diaphragm action of the floor system. Diaphragm action is essential for resisting and transferring horizontal forces to the vertical elements. Thus, diaphragm action is smaller for simple supported beams bearing on corbels than for beams cast monolithically to columns.

For resisting the forces induced by the earthquake, R/C frames and structural walls are commonly used A peculiar feature of this type of structures are the vehicle ramps. These form a vertical truss which braces the structure. Torsional movements may be induced in the building if the ramp is continuous

Half-dozen of parking structures failed. Seemingly, failures were due to low redundancy and high flexibility of the lateral load resisting system These buildings were designed to carry the gravity loads through interior columns which were not detailed to withstand cycling to large displacements (stirrup spacing was large, and in some cases, they were terminated in 90-deg. bends). In general, lateral load resisting system is placed around the perimeter of the building and is made of either ductile frames or ductile structural walls. Due to lateral displacement compatibility, assuming rigid diaphragms, interior columns were pushed to displacements that caused hinging at their ends. The combination of both non-ductile details and large horizontal and vertical accelerations explain the failure of the gravity load carrying system and the consequent structure collapse (Figs. 50 to 53). In some buildings, failure of short columns between ramps was recorded (Fig. 47).

All damaged parking structures did not have integrity reinforcement.

3.5 Steel Buildings

At the time of the reconnaissance reported herein, damage in non-structural components was the only type of distress reported. A number of curtain wall damages and fracture of interior water lines were reported. Water damage occurred in computer equipment, documents, archives, etc

Failures in welded flange-bolted web moment connections were observed in privately-owned buildings (the reconnaissance team learned about this problem after its return to Mexico) It has been reported in the literature that this detail may not provide satisfactory performance when used for beams in which the web accounts for a substantial portion of the beam's flexural strength. This observation has been attributed to the bolted web connection's limited ability to transfer bending moment, resulting in excessive demands on the beam-flange connections.

4. FINAL COMMENTS

The Northridge earthquake can be considered as a design level quake due to the magnitude of recorded accelerations. The vulnerability of buildings and bridges designed according to present code criteria can be assessed based on their performance. Given the similarities between the Mexico City Building Code and US codes, it is evident that Mexican engineers should be aware of research results.

Failures observed in this quake have pointed again the need for improving our design, detailing and construction practices. The good performance of some structures emphasize the idea of having simple codes whose requirements are applied rather than developing highly sophisticated codes which are not complied with.

The authors wish to thank Dr. Koji YOSHIMURA, Oita University, Japan, for providing us some pictures for this report.