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Impact of the 1989 Loma Prieta earthquake from a Canadian perspective

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ABSTRACT

Shortly after the earthquake, the Institute for Research in Construction of the National Research Council of Canada (IRC/NRCC) organized a team of engineers to visit the San Francisco Bay area. This paper gives an overview of the findings by the reconnaissance team. It covers important observations regarding the geotechnical aspects of the ground motion, the performance of building structures, liquid storage tank and other industrial facilities, electric power generating and distributing facilities, and transportation structures. A summary of the lessons learned from the Canadian perspective is also presented.

INTRODUCTION

On October 17, 1989, 17:04 Pacific Daylight Time, a strong earthquake, called the Loma Prieta Earthquake, of Richter magnitude $M_L = 7.0$ originated from the San Andreas fault and shook the entire San Francisco Bay area. The average surface wave magnitude (M_s) was estimated by the U.S. Geological Survey as 7.1. The epicenter was located approximately 16 km northeast of the city of Santa Cruz. The felt area stretched from Las Vegas, Nevada to Los Angeles, California. The hypocentral depth was about 18.5 km, which is relatively deep for most events associated with the San Andreas fault. The fault movement includes not only the typical horizontal component of slip, but also a significant thrusting of the southwest side up and over the northeast side. The rupture zone was 40 km long. The overall felt duration of the strong shaking in some areas reached 20 seconds.

The closest accelerographs were located in Santa Cruz and in Watsonville, and they recorded the highest horizontal and vertical peak accelerations: Santa Cruz – 0.64g H, 0.47g V; Watsonville – 0.39g H, 0.66g V. The records from San Francisco stations showed differences depending on the type of soil deposits and other geologic features. Figure 1 presents the measured peak horizontal acceleration vs. epicentral distance according to the strong motion data collected from the California Strong Motion Instrumentation Program network (Shakal et al. 1989) and the U.S. Geological Survey network (Maley et al. 1989). The attenuation tendency is also shown.

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This earthquake was one of the most costly natural disasters in U.S. history. The estimated cost of damage was in the range of \$6 – 10 billion. At least 62 people were confirmed dead and 3,757 people were injured. More than 12,000 people were displaced from their homes by the earthquake.

GEOTECHNICAL ASPECTS

Ground motion amplification and soil liquefaction were two major problems due to soil conditions during this earthquake. The amplification of earthquake motions is a complex process and is dependent on soil properties and thickness, frequency contents of motion and local geological settings. For a given earthquake and geological setting, the amplification increases with the increase of soil compressibility and thickness. In the San Francisco Bay region, various thicknesses of a soft deposit known as Bay mud are found. This deposit is normally underlain by a layer of alluvium. Measurements showed that these deposits amplified the maximum peak horizontal accelerations by a factor of two to three. Structures founded on these deposits therefore have been subjected to high horizontal excitation during the earthquake, hence resulting in many major damages in the region. Examples are the collapsed 2 km section of the Nimitz Freeway, the failure of one section of the Bay Bridge and many severely damaged buildings in the Marina District.

This earthquake caused extensive soil liquefaction damage in the form of sand boils, ground cracks, ground heave, building collapse and differential settlement. Many of these damages occurred in areas known to experience similar failures in the 1906 San Francisco earthquake. Figure 2 shows the locations of the damages as well as the corrected standard penetration resistance (N_1) of the soil, below which liquefaction will occur for this earthquake. The N_1 values were obtained from an energy approach for assessing liquefaction potential (Law et al. 1990). As an example, this method suggests that the Marina District will suffer liquefaction failure when N_1 is less than 10. Such a value is consistent with that of the hydraulic fill which liquefied there.

PERFORMANCE OF BUILDINGS

In general, the performance of buildings during an earthquake depends on a large number of factors such as: the type of structure, year of construction, lateral resistance system and local ground effects (soil liquefaction, densification of soil, differential settlement, lateral spread, uplift or rupture of actual fault line under the buildings).

Wood Frame Housing

The performance of wood frame housing in this earthquake has been reported by Bruneau (1989), and Jablonski et al. (1990) and Rainer et al. (1990). Two types of wood-frame houses sustained heavy damage. The first type is different variations of single-family dwelling in the epicentral area. The second type is multi-storey wood frame construction, typically 3 - 4 storey wood frame apartment houses or 2 - 3 storey townhouses in the Marina District in San Francisco (about 100 km from the epicenter). In the epicentral area, most of the damage was caused by failure of "crippled" stud foundation walls (also called "pony" walls). Cripple stud walls are common in older buildings in California. They are short (less that 14"

(35.6 cm) high), but may reach a height of one full storey in modern wood villas. Improper bracing and inadequate connections to the foundation and the upper-floor framing system caused many older buildings displaced from their foundations. In general, majority of wood houses performed well except those situated on large ground fissures. Outside the epicentral region, extensive structural damage occurred in the Marina District in San Francisco. The highest recorded acceleration nearby, 0.21 g, was recorded in Presidio on rock, a few blocks northwest from the Marina District. The garage floor of the typical townhouses and apartment buildings in the area behaved like a "soft storey" usually without bracing, or in some cases with only limited lateral resistance provided by sheathed walls (horizontal boards nailed to posts) (Fig.3).

Unreinforced Masonry Buildings

Many unreinforced masonry buildings (especially older ones) suffered severe damage and in some instances complete collapse. Examples of damage include out-of-plane collapses and large in-plane distress of the masonry walls, and brick veneer, bearing walls, parapets, and in-fill masonry walls. Severe damage to masonry buildings were reported in Santa Cruz (the Pacific Garden Mall), Los Gatos and Watsonville (Fig.4). Upgraded old stone masonry buildings at Stanford University campus in Palo Alto performed well.

Engineered Buildings

Case studies from the Loma Prieta earthquake related to engineered concrete structure has been reported by Mitchell et al. (1990). Most of the high-rise buildings in the San Francisco Bay area were not seriously damaged. There were some exceptions in Oakland, such as the steel frame building at Franklin and 18th Street, built in the early 1960's, which experienced significant shear cracks on the shear walls and some damage to the first floor columns.

PERFORMANCE OF TRANSPORTATION STRUCTURES

The Loma Prieta earthquake had a major effect on transportation routes and bridges in the San Francisco Bay Area. Two examples of the damage to transportation structures are presented here: the collapses of the Nimitz viaduct of I-880 and the failure of the Struve Slough Bridge on Highway 1 near Watsonville. The damage to the Oakland Bay Bridge has been described by Astaneh et al. (1989).

Collapse of Nimitz Viaduct of I-880

The collapsed freeway was located a short distance south of the east side entrance intersection of the San Francisco-Oakland Bay Bridge, approximately 100 km from the epicenter. The double deck freeway was designed in 1954 and constructed according to the design standards applicable at the time. There were two common types of design layouts, different mainly in the placements of pin connections in the upper frame structure. For most of the collapsed bents along the 1.2 km failed sections shown in Fig.5, the columns failed at the hinge joint regions due to shear force from the lateral load induced by the ground motion. Other details are reported by Jablonski et al. (1990) and others.

Struve Slough Bridge

The Struve Slough Bridge is actually two bridges for the north and south bound traffic. Due to large horizontal displacement associated with laterally shifted old river bed, some columns were sheared off and displaced from the external bridge girders as shown in Fig.6. The southern deck dropped about 30 cm down and many columns punched through it. The northern deck columns barely withstood the earthquake.

PERFORMANCE OF INDUSTRIAL FACILITIES

The San Francisco Bay Area is a major commercial and industrial center on the west cost of North America. Although many industrial facilities throughout the regions experienced strong ground motions from the main shock and the many aftershocks, in general most facilities survived the earthquake with only minor or no property damage.

Electric Power Generating Plants

The Loma Prieta earthquake disrupted the electric power systems to about 1.4 million customers throughout the regions. Within 48 hours, full service to most areas was restored except for about 26000 customers. Located about 30 miles (50 km) south of the epicenter is the Moss Landing Power Plant, which was damaged by the earthquake. Only the 750 megawatts natural gas fired unit 6 built in 1967–68 of this seven-unit plant, was operating at the time of the earthquake. The superstructure of unit 6, consisting of a 200 ft (61 m) high steel braced frame supported on the ground floor, generally performed very well. Only two bracing members buckled. There was no crack in the foundation slab, although some ground settlement was observed outside.

No damage to piping was observed. A vertical strut, supporting the horizontal snubbers for the super heater by-pass between the primary heater and secondary heater, yielded in bending as a result of the large displacement experienced during the earthquake.

The two 500 ft high steel lined, reinforced concrete stacks supported on pile foundations, did not suffer any serious damage, except for the cracking of the valve leading to the water tank located at the bottom of the stack.

In spite of the damage mentioned above, the plant was on-line again within 24 hours.

Substation Damage

The substations at Moss Landing, Metcalf and San Mateo, with epicentral distances of 45, 30, and 75 km respectively, suffered considerable damage. From the acceleration records of the surrounding areas, the intensity of the ground motions at those locations are estimated to be between 0.12 g to 0.45 g.

At the Moss Landing Substation, many seismic strengthened gas circuit breakers, designed to withstand about 0.30 g acceleration, failed and were overturned. The anchoring devices were inadequate, and the steel supports were dragged down by the failing circuit breakers. Other damage included the failures of several current transformers, disconnectors, wire-traps, and an oil leak in a power transformer. Similar circuit breakers failed at the 230-kV San Mateo Substation and the 500-kV Metcalf Substation because of rupture at the base of the porcelain column.

The total load loss due to this 7.1-magnitude Loma Prieta earthquake was estimated at about 4300 MW on the PG&E network with an overall system capacity of about 18000 MW. As a comparison, the Saguenay earthquake (M=6) caused a load loss of 3000 MW on the Hydro-Québec system, which has a capacity of 30226 MW.

Liquid Storage Tank Damage

Many thin-walled, stainless steel wine storage tanks in a winery facility, located approximately 20 km east of the epicenter, suffered earthquake damage. The severity of the damage depended on the liquid content level at the time of the earthquake. In general, empty tanks suffered little or no damage. In other cases, the anchored bolts of many tanks were pulled off the concrete pad of the foundation (Fig.7) as the tanks rocked in response to the sloshing liquid during the strong ground motions. Following the failure of the anchorage system, the high compressive axial stress from the rocking motion of the tanks buckled the tank shells.

At least two 25,000-gallon anchored, stainless steel fermentation tanks suffered the commonly known "elephant foot" type of buckling, of which the characteristic is that the buckling is generally located just above the tank base and spreads around the entire or most of the circumference Nearby, several 19,000-gallon tanks suffered the diamond-shaped type of buckling shown in Fig.8.

The liquid storage tanks in a food processing plant, located a few city blocks from downtown Watsonville, suffered similar damage as described above due to the earthquake.

LESSONS LEARNED FROM A CANADIAN PERSPECTIVE

One of the important characteristics of the Loma Prieta earthquake is the significant effect of soil conditions on the intensity of ground motions, leading to a very variable pattern of ground motions, and the many examples of liquefaction and liquefaction-induced damage on fills around the San Francisco Bay Area.

The experience of this earthquake indicates that the most serious deficiency in Part 9 of the NBC (1985) is the lack of any requirements for wall bracing in wood-frame construction, as well as the need for tying ends of beams over supports and anchorage of masonry chimney to the roof and floors. Lateral collapse of foundation cripple-stud walls was also a serious failure mode. This is covered by Part 4 in the NBC lateral force requirement via Subsection 9.15.15 and Article 9.4.1.1. Many designers or builders, however, may not be fully aware of this.

It was again shown that conventional power transmission equipments would be susceptible to earthquake induced failures. The major tremor predicted for the Charlevoix-

Kamouraska area in Quebec could reach a magnitude of 7 to 7.5, which is similar to that of the Loma Prieta earthquake. The maximum rock acceleration predicted for the Charlevoix-Kamouraska area is 0.70 g, as compared to the maximum acceleration of 0.64 g recorded near the Lome Prieta earthquake epicenter. Thus from the experience of this earthquake, the estimated acceleration values for the Charlevoix-Kamouraska area are proved to be very realistic.

Finally, this earthquake strongly demonstrated that three conditions occurring together can create a major seismic disaster: a high population density, a significant earthquake and a thick deposit of compressible soil. These conditions can be found in some regions of Canada. The most prominent examples are the Fraser River Delta in British Columbia and Quebec City in the Province of Quebec. Both are highly populated and located in areas of significant seismicity and with thick layers of soft alluvial deposits. Seismic hazards are therefore real both in the western and eastern regions of Canada.

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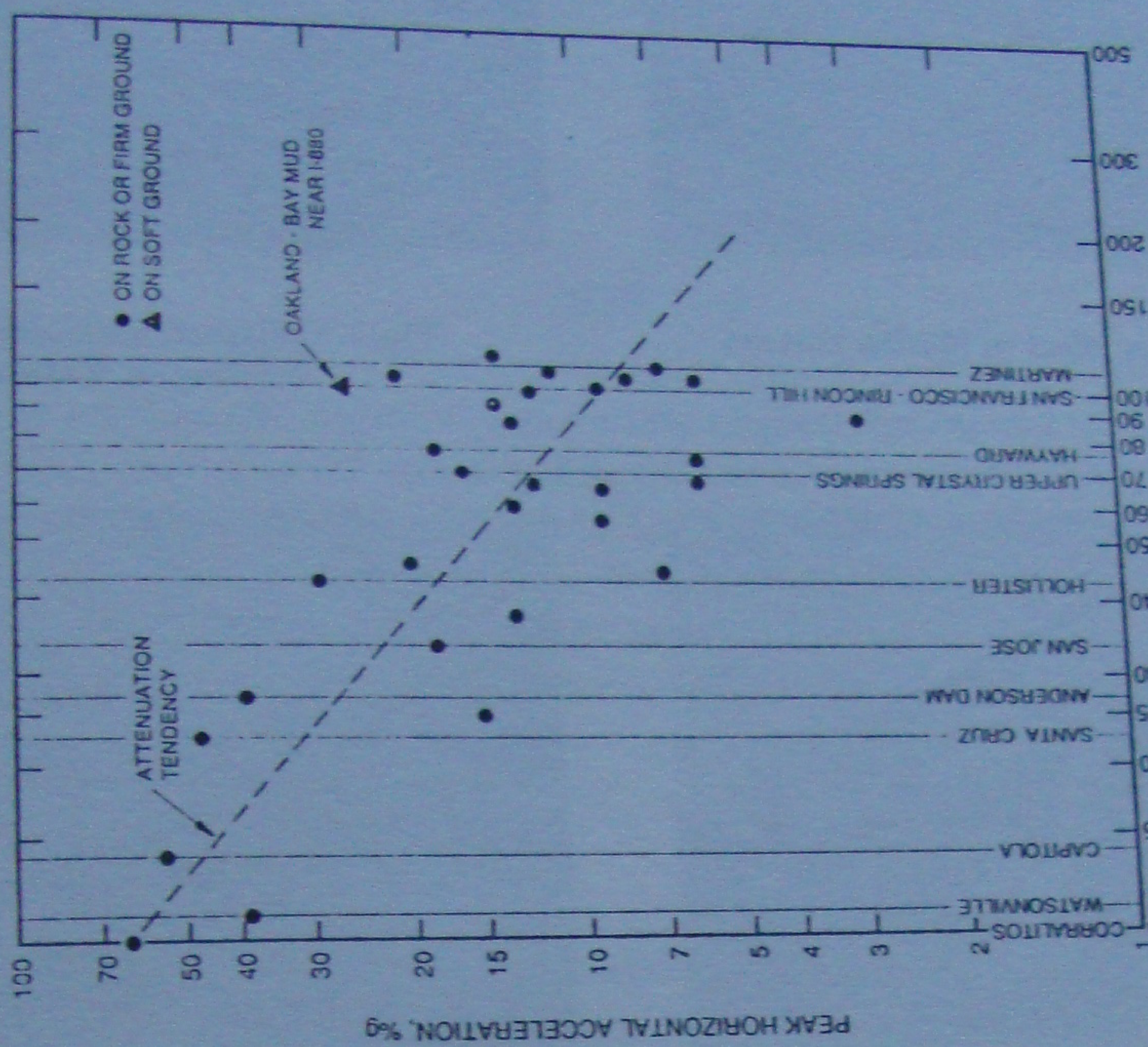

Fig.1 Measured peak horiz. accel. vs epicentral distance

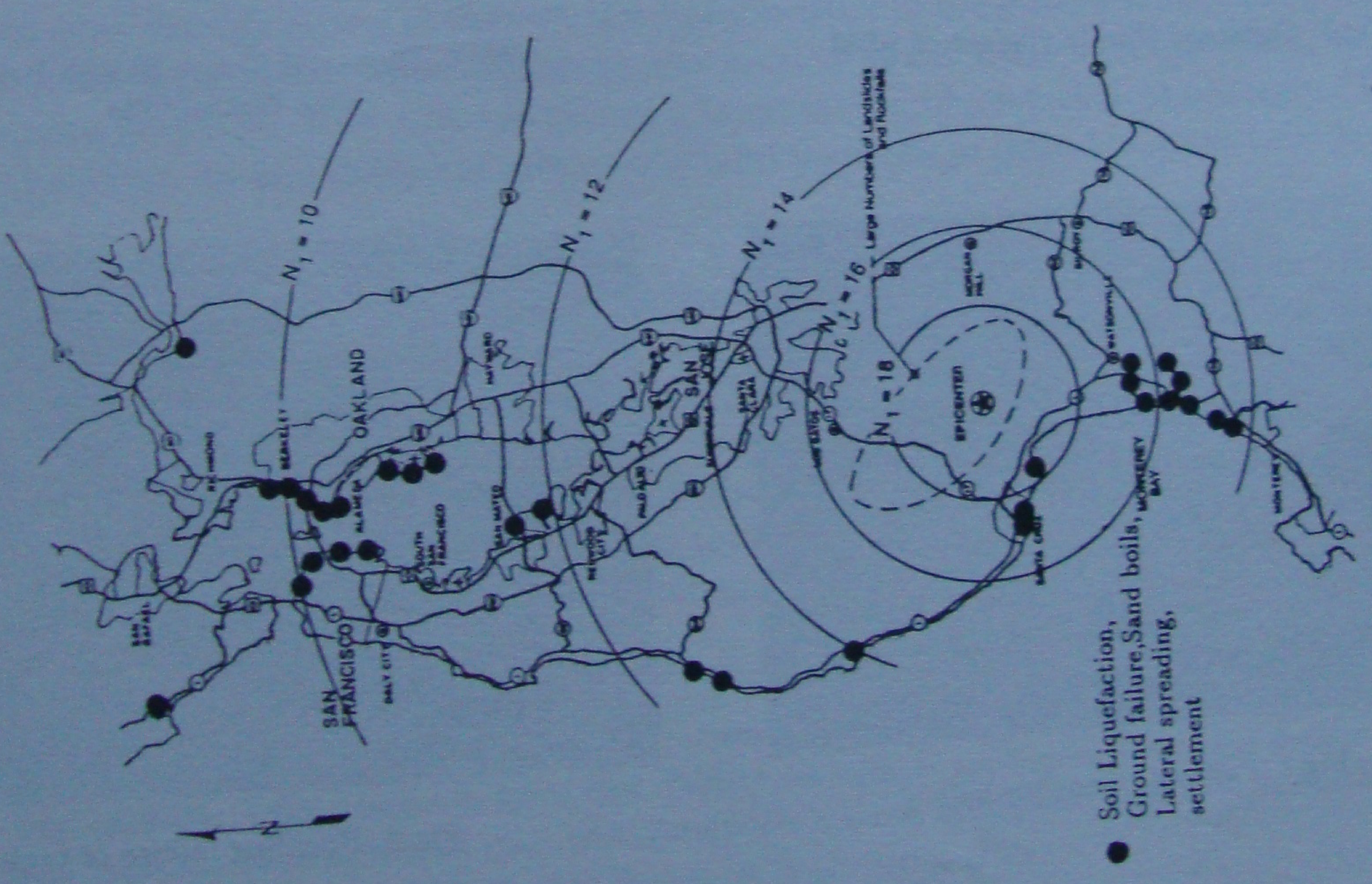

Fig.2 Location of soil failures and contours of standard penetration resistance (N_1) below which liquefaction will occur

Fig.3 Soft storey effect in Marina District

Fig.4 Collapsed unreinforced masonry building in Watsonville

Fig.5 Collapsed Nimitz viaduct of I-880

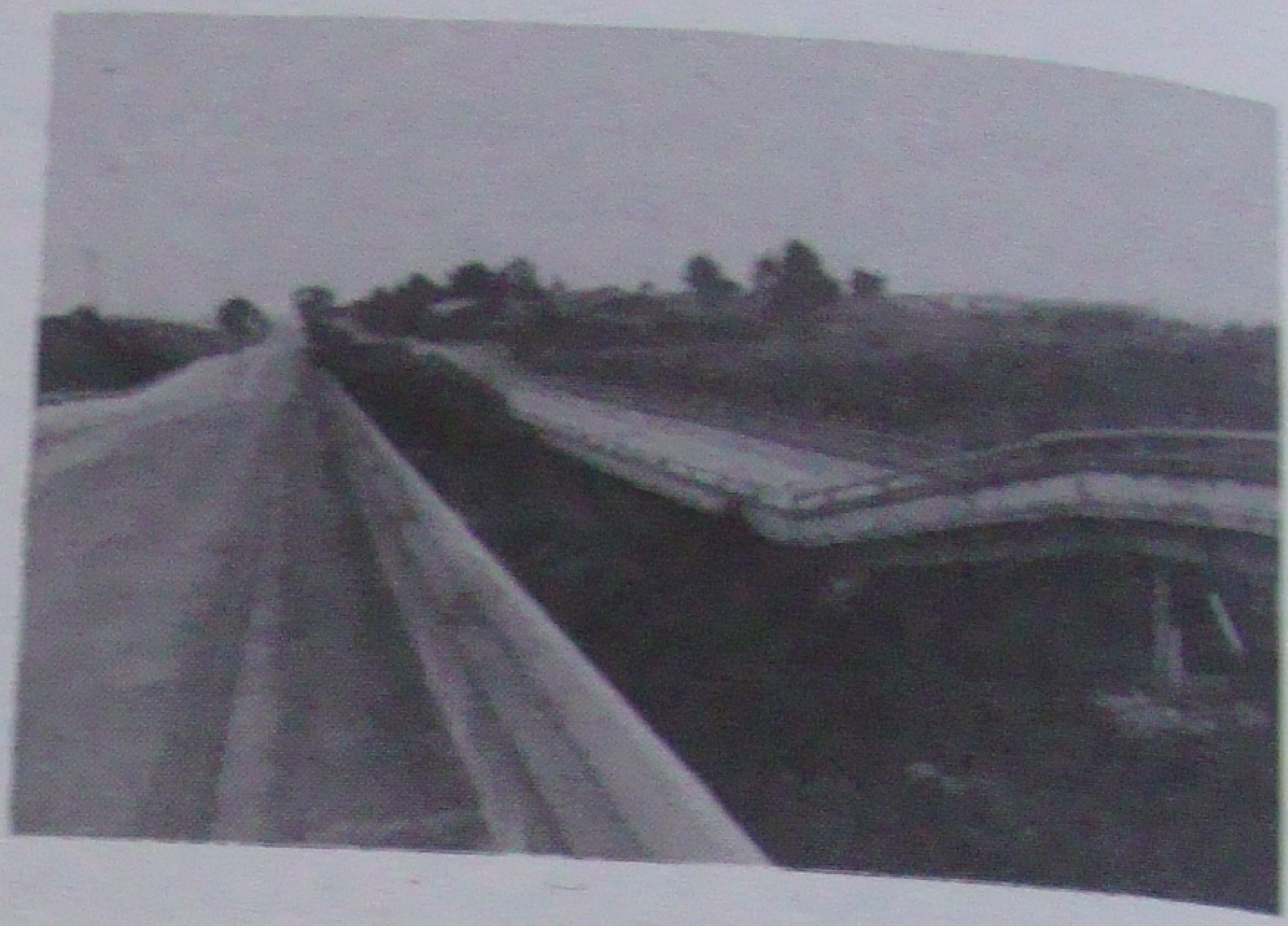

Fig.6 Damaged Struve Slough Bridge near Watsonville

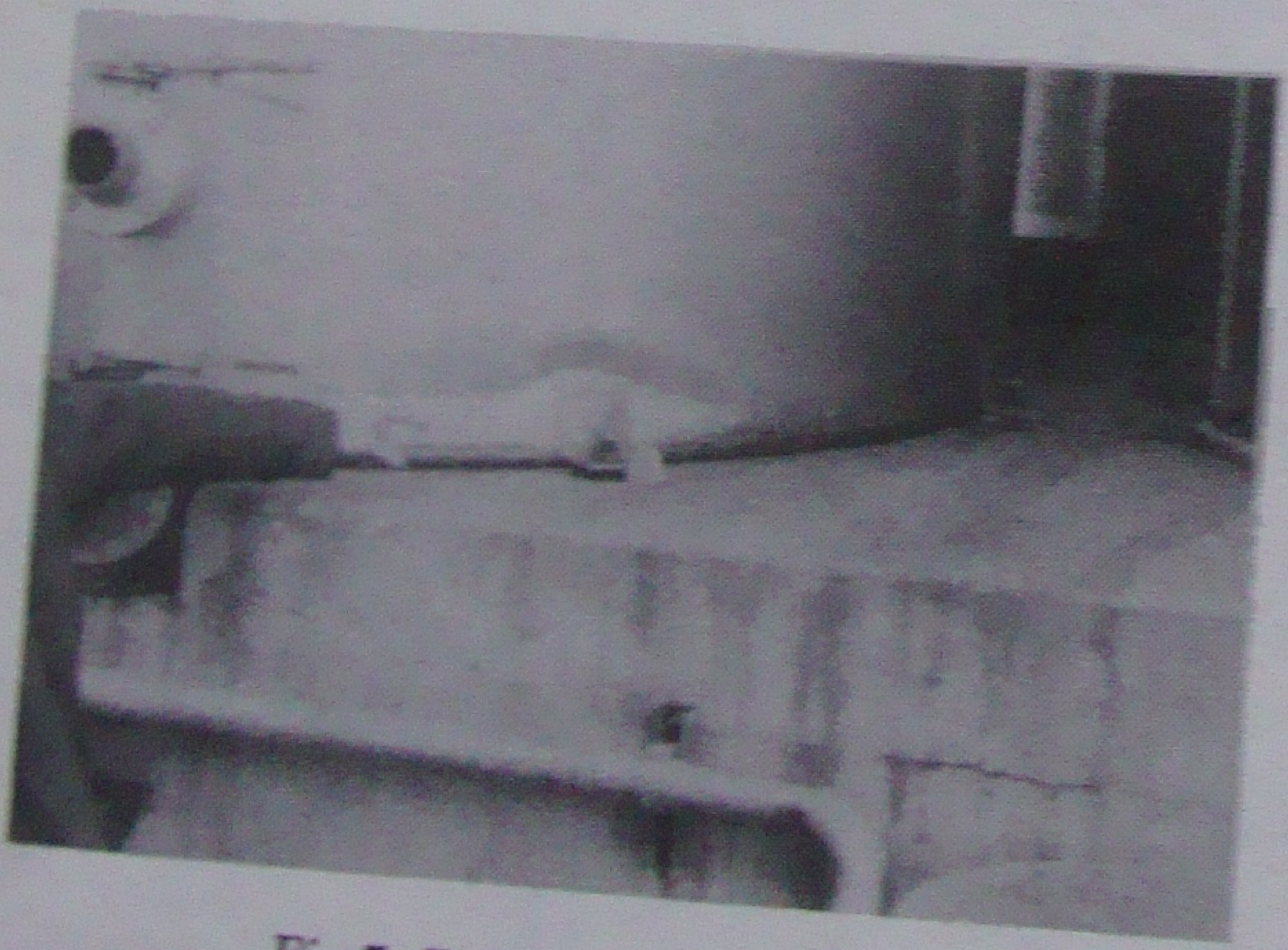

Fig.7 Pull out of anchor bolt

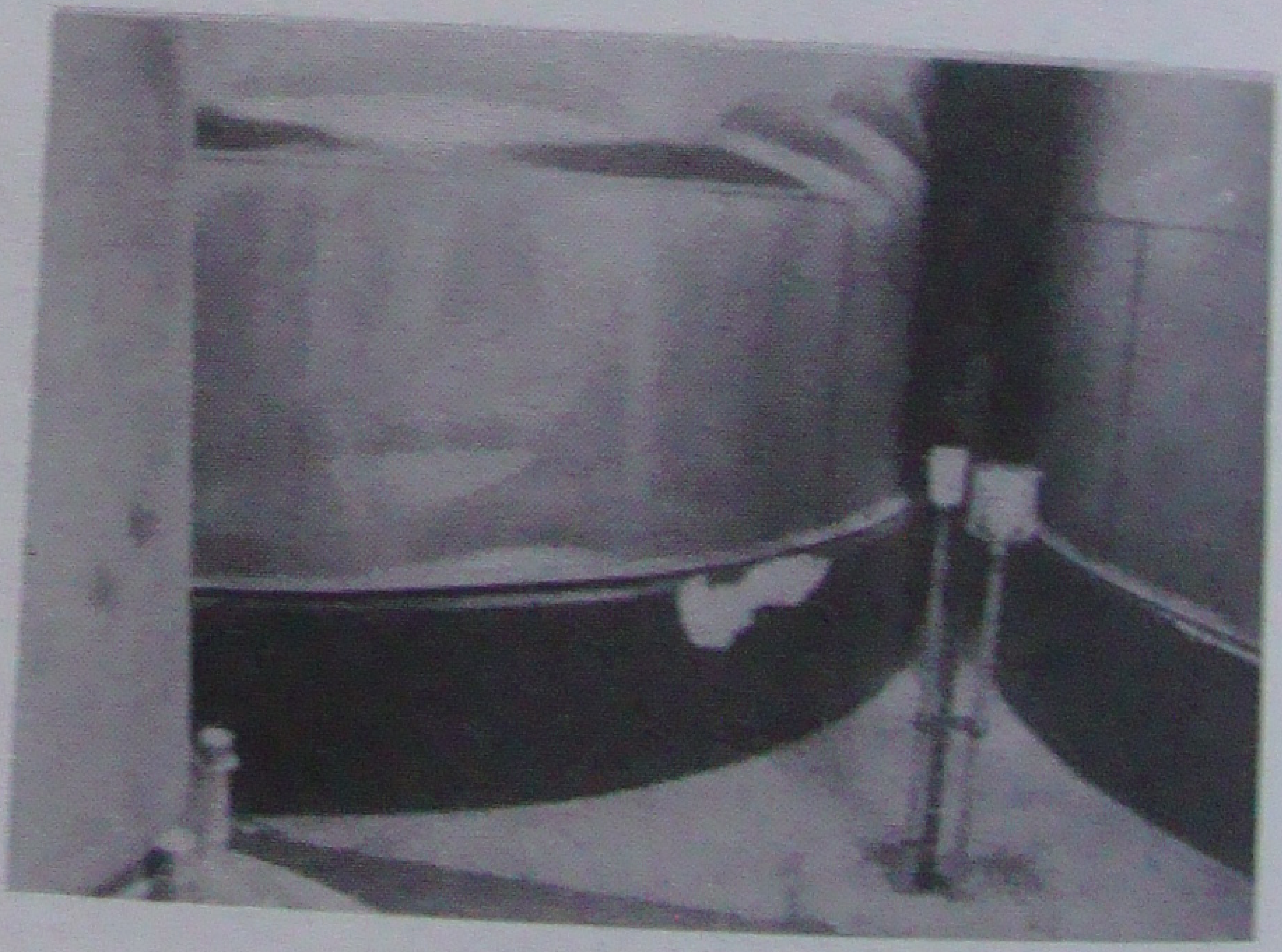

Fig.8 Buckling failure of tank