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The San Francisco area earthquake of 1989 and implications for the Greater Vancouver area[1]

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The earthquake that hit the San Francisco area on October 17, 1989, is reviewed with respect to damage to buildings, transportation facilities, and services. The San Francisco experience underlines that soil conditions and inadequate structural integrity are the two most important factors in the seismic risk to a building and its inhabitants. This earthquake is used as a model for the damage prediction in the Greater Vancouver area from a "design earthquake" that is implied in the National Building Code of Canada. In comparable housing density the expected damage would be somewhat greater than that observed in the San Francisco region in October 1989 because of differences in amplitude of ground motions and building design standards. This study is seen as a first step in the detailed assessment of damage potentials for the Vancouver region, or other similar metropolitan areas. Potential shortcomings in the 1985 National Building Code of Canada were identified in the seismic requirements for non-engineered buildings (Part 9) concerning lateral bracing, beam splice ties over supports, and anchorage and reinforcing of chimneys.

Key words: earthquake damage, building code, damage prediction.

Les auteurs étudient les dommages causés aux immeubles, aux équipements de transport et aux services par le séisme qui a frappé la région de San Francisco le 17 octobre 1989. Cet événement montre que les conditions du sol et le défaut d'intégrité des constructions sont les deux plus importants facteurs de risque sismique pour un immeuble et ses occupants. Les auteurs se servent de ce séisme comme modèle pour prévoir les dommages que causerait dans la région du Grand Vancouver un «séisme de calcul» comme celui qui est utilisé dans le Code national du bâtiment du Canada. Dans une région à densité de population comparable, les dommages seraient légèrement plus sérieux que ceux observés en octobre 1989 dans la région de San Francisco en raison des différences aux niveaux de l'amplitude des mouvements du sol et des normes de construction des bâtiments. Cette étude constitue la première étape d'une évaluation détaillée des risques de dommages pour la région de Vancouver ou une autre région métropolitaine semblable. On a relevé des lacunes possibles du Code national du bâtiment du Canada 1985 au plan des exigences sismiques visant les bâtiments non techniques (partie 9), plus précisément en ce qui a trait à l'entretoisement, aux plaques d'attache des poutres, vis-à-vis des supports, ainsi qu'à l'ancrage et au renforcement des cheminées.

Mots clés : dommages causés par les séismes, code du bâtiment, prévision des dommages.

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Introduction

On October 17, 1989, at 17:04 Pacific Daylight Time, a strong earthquake, called Loma Prieta, shook the entire San Francisco Bay area. Eight days after the earthquake, a team from the National Research Council of Canada consisting of two of the authors (Law and Jablonski) and three other participants visited the area. The prime objective was to determine the nature and extent of the damage to buildings and lifelines in the San Francisco Bay area with a view to predicting the expected impact of a similar magnitude earthquake in the Greater Vancouver area.

Such predictions should be of interest to emergency planners, community leaders, the design professions, and the population at large. Predictions are needed for making informed decisions on allocation of resources for preparedness, retrofitting of buildings and facilities, and for countering possible economic consequences of a destructive event. This study deals with the broad picture of the earthquake effects and is seen as a first step in the direction of detailed damage assessments for other locations; we hope further refinements will be made in the years to come.

Synopsis of Loma Prieta earthquake

The Loma Prieta earthquake of Richter magnitude $M_L = 7.0$ was caused by the rupture of a section of the San Andreas Fault with the epicentre located approximately 16 km northeast of the city of Santa Cruz (Fig. 1). Surface wave magnitude was calculated as $M_s = 7.1$ (USGS 1989). Within a period of 12 days after the main shock, 80 aftershocks of magnitude 3.0 and larger were recorded, the largest one being magnitude 5.2.

This was the largest magnitude earthquake in northern California since the San Francisco earthquake of April 18, 1906, of magnitude 8.3. The Loma Prieta earthquake has been estimated as one of the largest natural disasters in U.S. history, with $10 billion economic losses, 64 confirmed deaths, and more than 3700 injuries. It caused severe damage to a number of engineered structures: collapsed the Cypress Street viaduct of the interstate highway I-880 (also called Nimitz Freeway), where dozens of motorists were killed; collapsed the section of the San Francisco–Oakland Bay Bridge on the Oakland side of the bridge where the earthquake caused displacement of 18 cm; and heavily damaged a number of bridges

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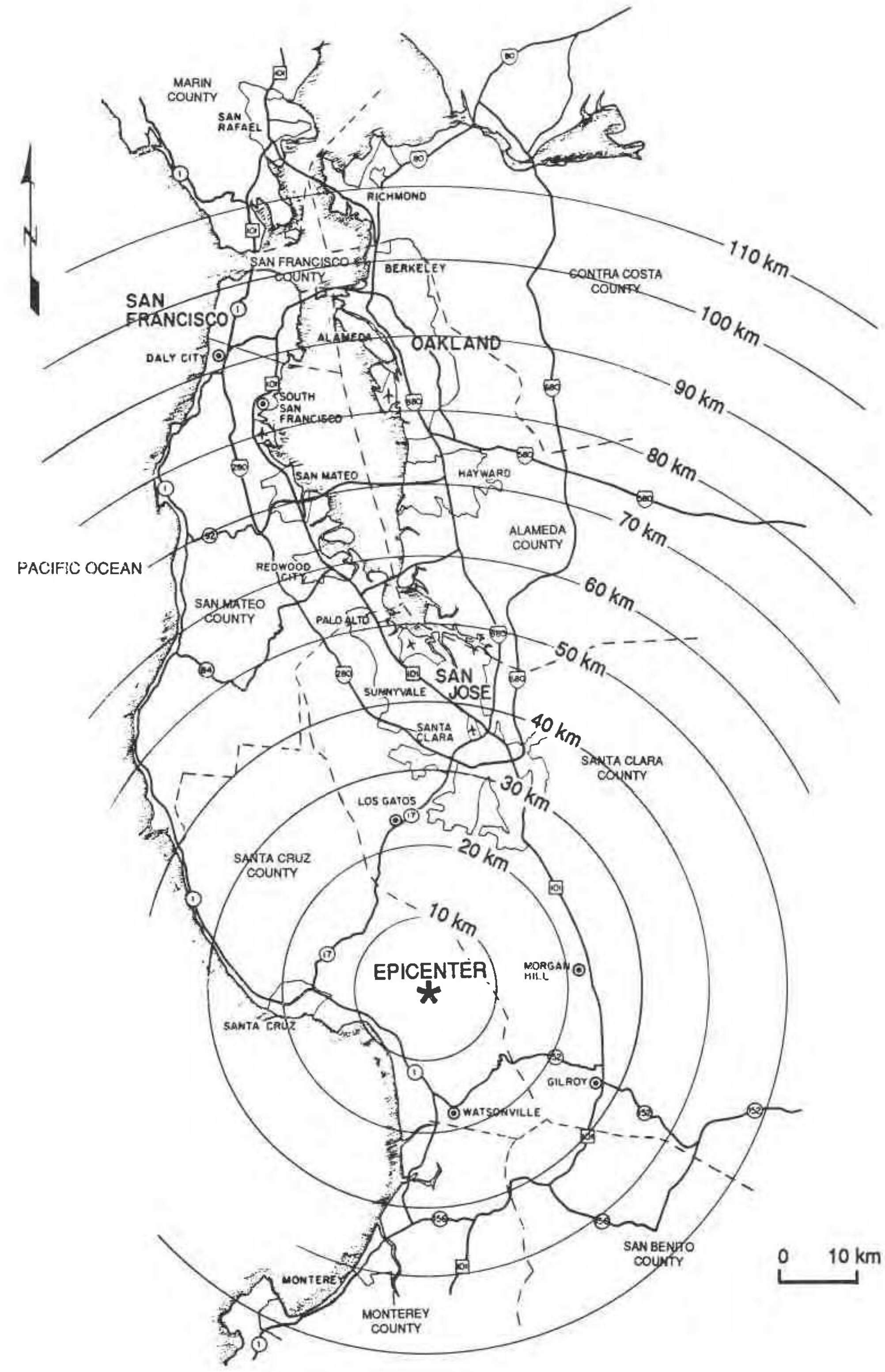

FIG. 1. The geographical areas affected by the Loma Prieta earthquake.

and highways. A number of cities were hit, including San Francisco and Oakland, but the major areas of destruction were limited to several pockets associated with soft soil deposits, especially fill areas. The earthquake caused ground failures in many areas, including soil liquefaction, landslides, soil lateral spreads, and ground cracks.

A preliminary overview of losses in the various counties of the San Francisco region is shown in Table 1 (expanded from Astaneh *et al.* 1989), along with the prevalent level of ground acceleration in the major built-up areas. The county boundaries are shown in Fig. 1. The number of buildings immediately condemned is seen to be around 500 outside of the epicentral area, Santa Cruz County.

A previous report on the Loma Prieta earthquake and per-

TABLE 1. Preliminary data on damage distribution (from Astaneh *et al.* 1989 and updated from county sources)

County	Fatalities	Injuries	Damage ($ billions)	Buildings condemned	Buildings damaged	Peak acceleration (% g)*
Alameda (Oakland)	40	349	1.5	29	3 411	10–20 (20–30)
Contra Costa	None	None	NA	NA	5	<10
Marina	None	None	NA	NA	5	<10
Monterey	1	None	0.05–0.1	45	144	5–10
San Benito	None	75	0.1	NA	NA	<10
Santa Clara	5	>650	0.65	>71	104 884	20–40
Santa Cruz	6	NA	1.0	580	3 290	40–70
San Francisco	13	NA	2.0	>350	NA	10–20 (20–30)

NOTE: NA = not available.

*Dominant level in major population centres on rock or alluvium. Values in parentheses are accelerations on soft ground and fill.

tinence to Canadian engineering practice has been presented by Bruneau (1990).

Strong-motion seismograph data

The Loma Prieta earthquake triggered over 100 strong-motion seismographic stations in the San Francisco Bay area. A preliminary summary of the strong-motion measurements is given by Maley *et al.* (1989), CSMIP (1989), and Shakal *et al.* (1989).

Figure 2 presents the peak horizontal accelerations vs. epicentral distance for a partial set of the recorded ground motions on rock or firm ground. The attenuation tendency is also indicated. High peak accelerations were recorded near the epicentre, at larger distances in Oakland near I-880, and at the Presidio (U.S. Army Base) in San Francisco, close to the heavily damaged area in the Marina District. A contour map based on the peak accelerations recorded on rock and alluvium is presented in Fig. 3, which shows that the peak ground accelerations attenuated rapidly in directions normal to the San Andreas Fault, but propagated with much less attenuation parallel to the fault. This could be due to different geological features such as the presence of numerous fault lines that parallel the San Andreas Fault, as well as source characteristics of the earthquake. This is also an explanation for the relatively large scatter of accelerations shown in Fig. 2, and indicates that a single attenuation rate is not always adequate for characterizing the seismological aspects of a site.

Effects of ground conditions

Widespread earthquake damage to structures and buildings is generally a direct result of the intensity and type of ground shaking. Local ground conditions can change the characteristics of earthquake motions that exist at the bedrock. In particular, thick deposits of compressible soils can raise the intensity of motions in a certain frequency range, leading to severe damage to buildings. Such deposits are abundant in the San Francisco Bay region and exist in three different types: fills, Bay mud, and alluvium.

Fills are man-made deposits normally loose in nature and much thinner than the natural deposits of Bay mud and alluvium. For example, in the Marina District, centrally located on the northern coast of the City of San Francisco, the fills were placed hydraulically. They are of very loose, uniform sand with sea shells. The *Bay mud* consists mostly of recent deposits (8 000 years and younger) of soft plastic carbonaceous clay, silt, and minor sand inclusions. It is loose, with moisture content normally exceeding 50%, and may be as thick as 40 m. Shear wave velocities in this deposit range from 90 to 130 m/s. The *alluvium*, with thickness reaching 600 m, corresponds to an older Bay sediment. It consists mostly of silty clay, silty clayey sand, sand, and gravel, It generally has a moisture content of less than 40%. The shear wave velocities in this deposit increase with depth and at the surface the value is about 200 m/s. The characteristics and distribution of Bay mud and alluvium are given by Borcherdt *et al.* (1975).

During the 1989 Loma Prieta earthquake these deposits responded in three ways, causing serious damage or collapse of structures and buildings: amplification of ground motions, liquefaction failure, and other ground problems such as settlement.

Amplification of motions

The amplification of earthquake motions depends on soil properties, thickness, frequency content of motions and local geological settings. For a given earthquake and geological setting, the amplification increases with increase of soil compressibility and with increase of soil thickness. The amplifications in the Bay mud are estimated to have been 2–3 times the bedrock values.

Thus, structures founded on these compressible deposits have been subject to high horizontal excitation during the earthquake. All the major damages in the City of San Francisco and a majority in Oakland occurred on these deposits. Many residential houses in the Marina District sustained severe damage and some even collapsed, a major factor being the high amplification due to the Bay mud and the hydraulic fill. Houses similar to the collapsed ones just outside the Marina District exhibited significantly less damage. The pier supporting the collapsed section of the Bay Bridge was founded on the Bay mud. Also, the collapsed Cypress section of the Nimitz Freeqay was built on compressible soil, while the noncollapsed section was founded on alluvium. This suggests that Bay mud yields a higher amplification than the alluvium. On the other hand, a number of multistorey steel frame buildings and reinforced concrete buildings in downtown Oakland suffered structural damage where alluvium prevails. Strong-motion records on alluvium also indicate amplifications compared to bedrock. In downtown Santa Cruz, where land was reclaimed with man-made fills, 85% of the unreinforced masonry buildings were damaged, although in this epicentral area the intensity of shaking was also substantially larger (Fig. 3).

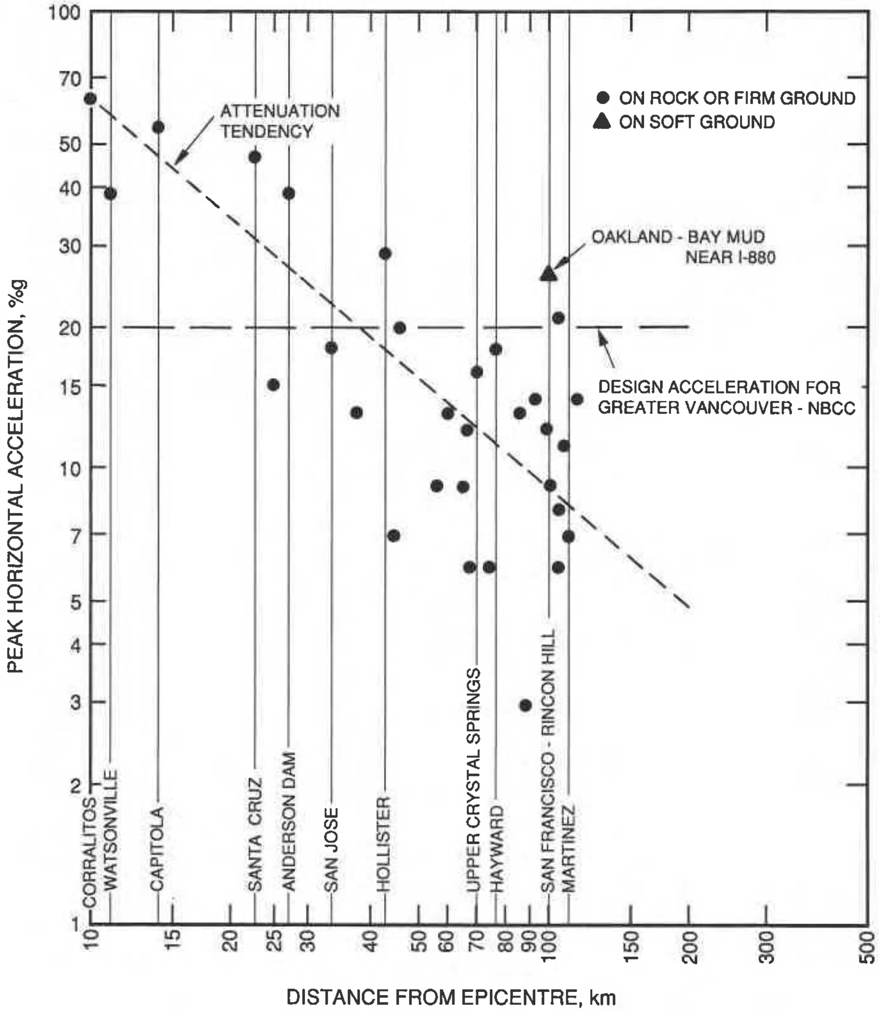

FIG. 2. Measured peak horizontal accelerations on rock or alluvium vs. epicentral distance, Loma Prieta earthquake (Data from Maley *et al.* 1989; CSMIT 1989; Shakal *et al.* 1989).

Liquefaction

Liquefaction is a process in which soft saturated granular soil is transformed to a liquid as a result of earthquake shaking or by other dynamic disturbance. When this happens the soil beneath the surface loses strength and, under pressure from the overburden, tends to eject the water and soil mixture through the ground surface. This will result in cracking of ground, ground heave, sand boils, and differential settlement. All these phenomena were observed over an extensive area ranging from very near the epicentre to more than 100 km away.

In the Marina District, liquefaction failure was widespread. More than 20 sand boils were noted by the visiting team (e.g., Fig. 4). Differential settlement, bearing capacity failure, pavement damage, and buckled sidewalks were observed (Fig. 5). Buried utilities, including gas lines, were broken and led to spectacular fires. The material that flowed to the surface was a dark grey uniform sand with occasional sea shells, indicating that the hydraulic fill placed on site liquefied during the earthquake.

Other ground problems

Loose granular deposit, both saturated or unsaturated, may densify, leading to considerable settlement even without the phenomenon of liquefaction. An example is found at Embarcadero Freeway on the northeastern coast of San Francisco. Here fills were placed on top of Bay mud. The structure of the freeway was damaged to the point of near collapse. The footings for the structures and nearby buildings founded on pile-foundations suffered from different degrees of permanent settlement. The paved ground surface at one location settled 15 cm because of densification of the fill. Subsidence was also observed at a number of bridge approaches.

A large number of landslides and rockfalls were reported in the Santa Cruz mountains near the fault rupture zone. Many of these landslides were partly caused by rain that came after the earthquake. Highway 17, one of the two main highways into Santa Cruz from the north, was closed. A number of single-family houses were destroyed.

Signs of distress in a number of dams were reported. The Lexington earth dam suffered from some cracks, as did about 1.5 km of the San Lorenzo levee in Santa Cruz. The abutment of the Elsman dam sustained some cracks. Another 1.5 km levee along the Pajaro River outside Watsonville was damaged with evidence of liquefaction failure.

Performance of buildings

The Loma Prieta earthquake provided an opportunity to test various types of building structures, from single-family dwelling houses and medium size buildings in the Santa Cruz area

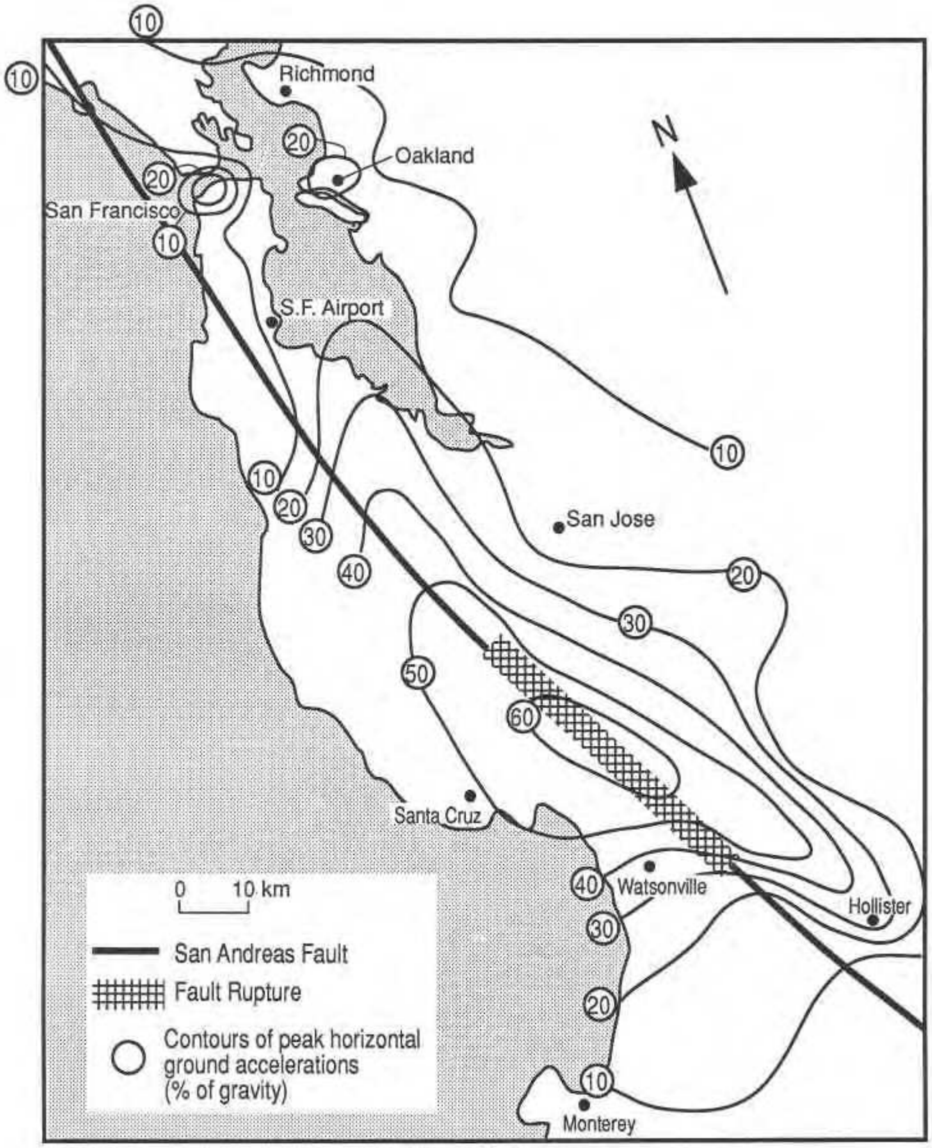

FIG. 3. Contours of peak ground accelerations recorded on rock or alluvium, Loma Prieta earthquake (Data from USGS 1989).

to high-rise buildings in San Francisco and Oakland. Structural damage was concentrated in pockets and depended on a number of factors, including type of the structure, year of construction, lateral resistance, and local ground effects. Aftershocks have augmented damage in several buildings. A brief description of representative damage patterns is presented.

Wood frame housing

Two types of wood frame houses sustained heavy damage: single-family dwellings in the epicentral area (about 50 km radius) and old wood frame apartments and townhouses in the Marina District in San Francisco (about 100 km from the epicentre).

In the epicentral area (e.g., in Watsonville and in Los Gatos) "cripple" stud foundation walls (or "pony" walls) failed in many old wood frame houses, causing serious damage. Cripple stud walls form the connections between foundations (concrete or masonry) and the first-floor framing. They are usually short, but in some modern wooden houses could reach a height of close to one full storey. Improper bracing and inadequate connections to the foundations as well as to the first-floor framing caused older buildings to be moved laterally off their foundations. The cripple walls were in some cases laying flat on their side (Fig. 6) where the acceleration reached $0.40g$ for about 16 s. Nailing on some failed walls was sparser than required by the building code. Also, large openings (e.g., porches) collapsed owing to lack of lateral resistance and settlement of the foundations.

The majority of modern wooden houses performed well unless they were situated on ground fissures. However, some houses with large openings like garage doors or with other irregularities sustained substantial damage in Los Altos Hills near Palo Alto, about 50 km from the epicentre, where the acceleration reached about $0.38g$. In general, poor connections or lack of structural continuity in the design were the prime reasons of damage in wood frame houses.

The area of greatest damage outside the epicentral region was concentrated in the Marina District in San Francisco situated on the fill placed after the 1906 earthquake. The highest acceleration on the nearest firm ground, $0.21g$, was recorded

FIG. 4. Sand boil, Marina District, San Francisco.

FIG. 5. Buckled pavement, Marina District, San Francisco.

in the Presidio (U.S. Army Base), a few blocks northwest from the Marina District and 105 km from the epicentre. The wood frame houses, constructed in the early 1920s, consist of two main types: two- and three-storey townhouses with garages on the street level, and three- and four-storey apartment buildings also with street-level garages. There is almost no separation between buildings. Many older three- and four-storey apartment houses situated at the street corners were badly damaged (Fig. 7). Some spectacular collapses of the entire building occurred when the one or two storeys were completely leveled. The garage floors had acted as a "soft storey" that appeared to have only limited bracing or none at all, or had sheathed walls constructed with boards nailed to posts. External stucco, brick, or fake stone walls were severely damaged and some collapsed. Many two-storey townhouses within the blocks also sustained some damage over garage doors and in walls. Entire blocks of buildings apparently responded together during the earthquake.

For some buildings where more horizontal resistance is provided by sheathed walls, the damage was much smaller. In

FIG. 6. Collapsed cripple wall and porch in a residence at Main Street in Los Gatos.

FIG. 7. Near collapse of a four-storey wood frame apartment building at the corner of Beach and Broderick streets, Marina District, San Francisco.

general, upper floors sustained little or no structural damage, although the entire ground floor of the building may have shifted. In other locations, houses at the interior of the block suffered less damage than those at the end of the block.

Unreinforced masonry buildings

Unreinforced masonry buildings near the epicentre built at the turn of this century suffered severe damage or collapsed (Figs. 8 and 9). Out-of-plane failures of upper portions of walls and of parapets were common. In some cases the severe shaking at the roof level resulted in separation of the roof from the walls. The result was not only the collapse of upper portions of walls but also of the roof structure, and this inflicted heavy damage to lower floors. Other old unreinforced masonry buildings in Oakland were heavily damaged.

Upgraded unreinforced brick masonry buildings performed well. Upgraded stone masonry buildings at Stanford University in Palo Alto also performed well, in contrast to unrein-

FIG. 8. Collapsed roof caused damage to the lower floors in the unreinforced masonry building on Campbell Street in Oakland.

FIG. 9. Major damage to unreinforced masonry wall building, 39 Main Street, Los Gatos.

forced masonry and old-style reinforced concrete buildings that suffered damage estimated at over $100 million; substantial structural damage was sustained by the masonry walls of the chapel.

Engineered buildings

The high-rise buildings in downtown San Francisco and Oakland rode out the earthquake without serious damage to the structural frame or the functionality of the buildings. But problems were encountered with elevators and with breakage of glass panes that showered debris onto the street. The pyramid-shaped Transamerica Building in downtown San Francisco received ground motions of $0.11g$, whereas the 49th floor near the apex recorded $0.31g$. It should be noted, however, that the ground motion experienced by the buildings in downtown San Francisco is about one half to one quarter those of the "design earthquake" for that location. Consequently, they were not tested to the full extent of their intended capacity.

Closer to the epicentre, the Palo Alto VA Hospital Building 1 was subjected to $0.34g$ at the base, and responded with $1.09g$

at the 7th floor below the roof. The highest recorded horizontal acceleration in a building was in the four-storey Government Building in Watsonville, in which $0.39g$ was recorded at the base and $1.24g$ at the top storey (Shakal *et al.* 1989). Both these buildings suffered little damage.

Performance of services

Transportation structures

In addition to the collapse of more than 1.5 km of elevated roadway of I-880, and collapse of a 16 m span on the upper deck of the San Francisco–Oakland Bay Bridge, 13 of the 1 500 highway bridges in the area suffered major structural damage; 73 others suffered less severe damage (ASCE News 1989).

Damage to the control tower at the San Francisco international airport closed the facility for 13 h, and liquefaction and settling also forced a runway closing at the Oakland airport. The tunnels and tracks of the BART (Bay Area Rapid Transit) system, however, performed well with only short temporary disruptions of service.

Water and sewage

There was extensive damage to water lines from ground deformations. In San Francisco, 72 significant pipe failures occurred in the Marina District and 25 breaks outside that area. A break in a 30-cm (12-in.) high-pressure line south of Market Street, where there was significant liquefaction, caused depletion of a 3 410 000-L (750 000-gallon) tank used for fire fighting. Hollister reported over 100 broken water mains and Santa Cruz, over 60. Large water mains failed in Santa Clara County, Los Gatos, and in the East Bay Municipal District; the latter reported over 140 additional broken mains (ASCE News 1989).

Assessment of damage to sewage collection systems is more difficult, since they generally do not operate under pressure and will continue to operate even if leaks are present. Typically, however, sewage systems are more vulnerable to seismically induced differential soil movements than water systems because the former are made of more brittle materials.

Gas and power

Only three failures of gas lines were reported: leaks in a 51-cm (20-in.) semi-high-pressure welded steel distribution line in Oakland, a 30-cm (12-in.) line in Hollister, and a 20-cm (8-in.) line in Santa Cruz (ASCE News 1989). In the Marina District, about 16 km of gas lines will need to be replaced at an estimated cost of $20 million. These gas lines were made of cast iron. New lines were being installed in the same area using flexible plastic pipes according to ASTM 2513 specifications. Automatic shutoff valves for gas supply exist in California but they are generally unpopular, since they are prone to accidental closure and can be turned on only by qualified personnel. This has sometimes taken weeks during previous drills or small earthquakes.

Initial electric power outages affected about 1.4 million customers. Within 48 h, though, service to all but 26 000 of those had been restored. The most severe damage occurred to substations, primarily to ceramic members of circuit breakers and oil leaks to transformers. Major damage occurred at two key substations in San Jose and San Mateo, the 500-kV switchyards at the Metcalf substation and the 500-kV switchyard at Moss Landing. At least one distribution station in the epicentral area also had its transformers damaged (ASCE News 1989).

Communications

As in most earthquakes, an increase in telephone traffic in the hours immediately after the event overloaded the system so that there were long delays in getting dial tones on non-priority lines. Calls could be made within the same area code in most areas, however, if they were dialed several times. Radio announcements right after the earthquake requested that only emergency calls be made and this probably contributed to the system's overall good performance.

The use of cellular phones was singled out as the best performer at the control centres located in specific disaster areas. CB radios were completely jammed because of the overwhelming usage after the earthquake. Other telephone lines were in operation except those that went through sophisticated private switching units. Some of these units failed because of the power outage; apparently their standby power supply had not been maintained.

Elevators

California has a special elevator code for use in tall buildings in earthquake zones. The performance of many elevators built according to this code, however, was not up to expectation. The most common problem was that the counter weight jumped off the guide rail, rendering the elevator unusable. It appears that the code needs revising.

Emergency preparedness

California has made extensive preparations for the effects of earthquakes. Most California cities have annual drills for earthquake emergency. The drills are costly and inconvenient but they demonstrated their value in dealing with this disaster. Shortly after the earthquake, control centres were set up at the major disaster areas and were staffed by police, firefighters, rescue workers, building inspectors, and authorized volunteers. As one example, and as part of the emergency plan, volunteers from as far as Los Angeles were on their way to the San Francisco Bay area within minutes after the earthquake.

In the epicentral area, tent shelters were erected in public parks and food was provided by the American Red Cross. Structural engineers who had previously been trained in earthquake damage assessment categorized buildings into three groups: safe (designated by green posters), unsafe (red posters), and limited access (orange posters). Access was completely denied to "unsafe" buildings; entry into buildings declared "limited access" was permitted under supervision and for short periods only.

Basis for applying San Francisco experience to Vancouver

To estimate damage predictions for Vancouver on the basis of the experience with the Loma Prieta earthquake (and other similar earthquakes) requires consideration of the following factors:

- earthquake characteristics (magnitude, depth and type of rupture, duration, frequency content);
- location of earthquake, distance and direction from epicentre;
- geology and soil conditions;
- codes and standards;
- type of construction; and
- level of earthquake resistant design in use when buildings were constructed.

Damage addressed here is of a level that would prevent safe occupancy of the building immediately after the earthquake.

This can vary from repairable damage to total collapse. In the context of the California earthquake preparedness plan, this would include buildings tagged with "orange" and "red" posters in the post-disaster assessment.

Earthquake characteristics

The magnitude 7.1 Loma Prieta earthquake generated ground motions consisting of relatively low frequencies (judged to be mainly between 1 and 3 Hz horizontally, and 2 and 6 Hz vertically) and duration of shaking of about 10–15 s. These are typical values for moderate earthquakes that occur on the west coast of California. By reasonable extrapolation, a similar type of earthquake can be expected for Vancouver, at an epicentre within 50–80 km of the city and produce a ground acceleration in the Vancouver area that corresponds to the 0.20g design acceleration of the National Building Code. Earthquakes with a magnitude 6 at epicentres of 30–50 km from the city would have similar effects, but the duration might be somewhat less. On the other hand, earthquakes larger than magnitude 7.0 at distances of 150–200 km could be expected to produce similar ground motion amplitudes but with longer duration. Duration is an important parameter as it pertains to severity of damage. Thus for comparison purposes, an earthquake of similar magnitude and distance from the epicentre is the most suitable one for estimating damage in Vancouver. Not considered here is the possibility of the "subduction earthquake," with possible magnitudes up to 9.3 and duration of minutes (Rogers 1988).

Frequency content is a function of the rupture mechanism, earthquake magnitude, distance, and geologic features in the affected area. It can reasonably be assumed that frequency content in a future earthquake near Vancouver is not significantly different from that of the San Francisco area ground motion.

Soil conditions in the Greater Vancouver area

The geology of the Greater Vancouver area is described by Blunden (1973) and by Byrne and Anderson (1987). The area is underlain by thick clays, followed by sands, silt–clay deposits on the bedrock, or bedrock itself in some areas north of the Fraser River. There is a certain variability in soil conditions between the Fraser Delta and Burnaby Ridge in the north–south direction and also between the Fraser Delta and the eastern part of Surrey.

Typically, the Fraser Delta deposits have the following layers: (1) a surficial deposit comprised of thin layers of clays, silts, and peats (max. thickness of 8 m); (2) sand deposits (about 45 m); (3) silt–clay deposits (about 200 m); (4) glacial deposits (about 100 m); and (5) bedrock. The water table in the lower areas is generally within a metre of the ground surface.

The western portion of the Greater Vancouver area, except the central hilly part of the municipality of Surrey, is generally underlain by silty–clay deposits, while there are peat deposits in the eastern portion. The liquefaction threat is mainly for thick layers of sand deposits underlying thin crust of clays or silts. The dynamic liquefaction resistance of these sands can be estimated from their standard penetration resistance value, N, as will be discussed later.

Conditions for liquefaction

Liquefaction will occur when the induced dynamic stress exceeds the dynamic strength of the soil. The dynamic stress is a function of the earthquake magnitude and epicentral distance as well as the geometry and mechanical properties of the soil deposit. On the other hand, the dynamic strength is a function of the soil type and duration of shaking. There are a number of methods of expressing the dynamic stress and the dynamic strength. The method presented by Seed *et al.* (1983) contains the essential concepts and has been widely used. Based on this method, the dynamic stress, τ_h, is given by

$$[1] \quad \tau_h = 0.65\frac{a_{max}}{g}\sigma_v r_d$$

where a_{max} = peak horizontal acceleration at ground surface, g = gravitational acceleration, σ_v = total vertical stress, and r_d = a reduction factor varying with depth. The value of a_{max} depends on a number of factors including the spectrum of earthquake waves, attenuation property of the bedrock, and the amplification due to the soil deposit. The dynamic strength, τ_l, was established by Seed *et al.* (1983) based on observations of actual earthquakes around the world. It is expressed in terms of normalized standard penetration test (SPT) resistance, N_1, and a coefficient, μ, related to earthquake magnitude. A general form of the expression is given by

$$[2] \quad \tau_l = \mu f(N_1)$$

where $f(N_1)$, a function of N_1, is different for sand and for silty sand. Liquefaction will take place when the dynamic stress, τ_l, exceeds the dynamic strength, τ_h:

$$[3] \quad \frac{\tau_l}{\tau_h} \leq 1.0$$

Liquefaction potential

Liquefaction potentials were assessed for the deposits in the Fraser River delta. There are other soft deposits in the area, however, and the same analysis as presented here should not be assumed to apply unless specifically checked. The present analysis is based on the design earthquake according to the National Building Code of Canada 1985 (NBCC 1985), with a return period of 475 years and a peak bedrock acceleration, a_m, of 0.20g. The corresponding design earthquake magnitude, M, as suggested by Byrne and Anderson (1987) is taken as 7.0.

The induced dynamic stress, τ_h, depends on the maximum ground surface peak acceleration, a_{max}, which is a function of the bedrock acceleration, a_m. A study by Byrne and Anderson (1987) on typical soil profiles from Richmond shows that either slight amplification or slight deamplification is possible when the seismic waves travel from bedrock through the soil to the ground surface. It is assumed, therefore, that there is no change in amplitude of the acceleration from bedrock to the ground surface, i.e., $a_{max} = a_m$. The validity of this assumption, however, would need to be confirmed by measurements of seismic response at the surface and on nearby rock or at great depth.

The dynamic strength of the alluvial deposit can be obtained from the standard penetration test (SPT). Figure 10 shows profiles of regular SPT resistance, N, at various sites in Richmond and the Fraser Delta (Byrne and Anderson 1987). N can be normalized by the confining pressure to yield N_1 for application in [2]. Some soil variability can be clearly identified and, therefore, in order to assess the dynamic strength accurately, each site has to be studied separately. It is, however, revealing to consider some typical profiles to give a broad picture of the liquefaction potential for this region. Three profiles are chosen for this purpose: the mean, the lower bound, and the upper bound (Fig. 10). The mean corresponds to the average of all

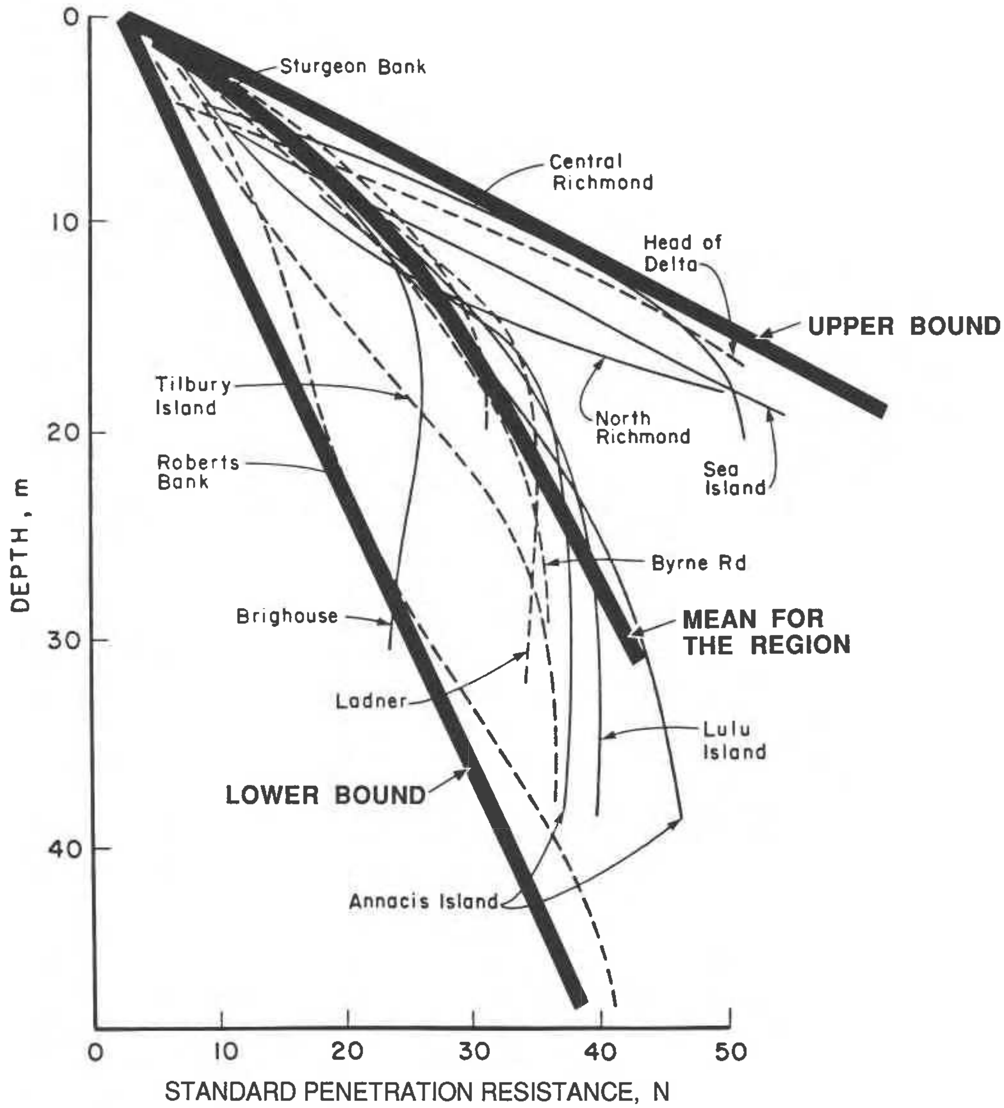

FIG. 10. Mean standard penetration resistance values, N, and derived bounds in Richmond, B.C., and Fraser Delta (Data from Byrne and Anderson 1987).

the profiles while the lower and upper bounds correspond to the weakest and strongest profiles. From these profiles, the dynamic strengths are obtained using [2]. Since both sand and silty sand exist in this region, strength profiles for both soil types have been obtained.

The liquefaction potential of the deposits from the Fraser Delta can now be studied by comparing the dynamic strength, τ_1, and dynamic stress, τ_h. The results are shown in Fig. 11, which show that for the design earthquake, sand and silty sand with average strengths are expected to liquefy to depths of 8 and 5 m, respectively.

Codes and standards

Two major types of construction can be recognized: (*i*) engineered construction and (*ii*) residential "non-engineered" construction. For purposes of comparisons, the 1985 versions of the National Building Code of Canada (NBC) and the Uniform Building Code (UBC) will be used. These codes provide the basis for applicable regulations in Vancouver and San Francisco area, respectively.

Engineered buildings (Part 4 of NBC)

Engineered construction follows applicable building codes as a minimum, but these standards are often exceeded for special structures such as some tall buildings. Most buildings in the San Francisco area would be designed to a zone 4 requirement in the UBC. In the Vancouver area the NBC requirements correspond to those of a velocity and acceleration zone $Z_a = Z_v = 4$ (no relation to the UBC zone 4) with $v = 0.20$. Comparable types of buildings and therefore comparable coefficients in the specified lateral forces are used for this comparison. On soft soil deposits, a foundation factor up to 1.5 is applicable, but the comparison presented here will be made for "rock or firm ground."

For the San Francisco area, the 1985 UBC design base shear for low-level buildings results in a base shear, V, of $0.12W$, where W = weight of building. For a comparable building in Vancouver, the 1985 NBC prescribes a base shear, V, of $0.088W$. For tall buildings (example period, T = 2 s), $V_{UBC} = 0.047W$ and $V_{NBC} = 0.031W$. Since both codes utilize a load factor of about 1.5, it can be seen that the ratio of

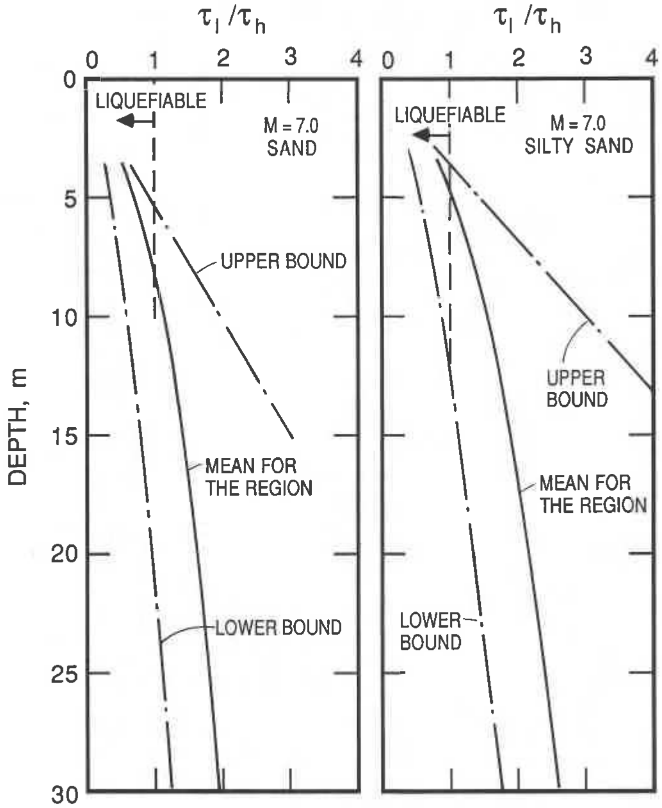

FIG. 11. Derived liquefaction potential of alluvial deposits in Fraser Delta during a design earthquake with $a_{max} = 0.2g$ and $M = 7$.

design forces for Vancouver to those of San Francisco is about 2/3. Thus the design forces in NBC 1985 for Vancouver would correspond to slightly less than a zone 3 requirement in the UBC, which in turn is 3/4 that of San Francisco. It should be pointed out that neither the NBC nor the UBC addresses the problems of liquefaction directly, but both codes consider it an area deserving special attention. The topic is, however, treated in the Canadian Foundation Engineering Manual (CGS 1985).

Non-engineered buildings (Part 9 of NBC)

Non-engineered buildings (also called "residential construction") include single-family houses and multi-unit dwellings up to and including three storeys in height or 600 m^2 of floor area.

Table 2 compares the requirements for residential construction governed by Part 9 of the NBC with the UBC. The requirements are similar except that some earthquake requirements in the UBC are missing from the NBC. Where requirements exist in both codes, they are essentially the same. The experience of the Loma Prieta earthquake indicates that the most serious deficiency in Part 9 of the NBC is the lack of any requirements for wall bracing in wood frame construction. Ground floors of two- or three-storey residential buildings containing large openings are vulnerable.

Other potential deficiencies in Part 9 of the NBC include the need for tieing ends of beams over supports and anchorage of masonry chimneys to the roof and floors. Collapses due to these deficiencies have occurred in this and previous earthquakes. Lateral collapse of foundation walls weak in racking resistance (such as cripple-stud walls) was also a serious failure mode near the epicentre. This is covered by the NBC Part 4 lateral force requirements via Clauses 9.15.1.5 (wood frame foundations) and 9.4.1.1 (construction not specified in Part 9). Many designers or builders, however, may not be aware of this.

Nearly all the serious structural failures that occurred to residential construction in the San Francisco area were due to deficiencies that are prohibited by the recent issues of the UBC. Although the earthquake intensity was less than the design earthquake except near the epicentre, the experience indicates that the present UBC requirements appear satisfactory. An exception to this is that the veneer anchor ties failed. Since these are specified in 3006(d)1 of the UBC and since Part 9 of NBC has a similar requirement, the detailed reasons for the failures should be investigated.

The main problem in Canada, as in other places, concerns the safety of existing buildings with serious deficiencies, such as unreinforced masonry, non-ductile concrete, and wood frame complexes with weak ground storeys. The seismic evaluation and upgrading of existing construction therefore requires attention.

Level of earthquake resistant design

For engineered construction, the level of earthquake resistant design that was employed when the buildings were constructed also plays a role in comparing damage potential. As was pointed out above, the seismic requirements for engineered construction are lower in Vancouver than in San Francisco. Therefore the same earthquake would be expected to produce more damage in the Greater Vancouver area.

For non-engineered construction, nominal lateral resistance in the UBC is achieved by specifying minimum percentages of shear panels in the walls. Since no such requirement is contained in the NBC, Part 9, it can be concluded that the lateral resistance of houses with weak configurations (e.g., large openings) in Vancouver is likely to be less than in the San Francisco area, thus making these buildings more vulnerable to comparable size earthquakes. The level of awareness among builders of potential earthquake hazard is also likely to be somewhat higher in California than in Canada, again pointing to a possible lower level of overall seismic resistance for houses on the British Columbia coast. The lateral resistance in most wood houses is inherently quite high, however, and therefore the overall reduction in seismic resistance should be marginal.

Damage assessment for the Vancouver area

Extension of the San Francisco area earthquake to ground motions that correspond to the design earthquake for Vancouver requires a number of assumptions and extrapolations. As a rough approximation, the peak ground accelerations recorded on firm ground from a particular earthquake can be compared on a par with the specified NBC design acceleration, which is $0.20g$ for Vancouver. The San Francisco region was subjected to maximum ground motions on firm ground ranging from about three times that value ($0.64g$) in the epicentral area, down to about one half ($0.10g$) in parts of Oakland and the City of San Francisco (Fig. 2). Since the epicentre is assumed to be outside the densely populated regions, a range from $0.30g$ to $0.10g$ would apply for Vancouver, i.e., straddling the "design" acceleration of $0.20g$. A major subduction earthquake (Rogers 1988; Heaton and Kanamori 1984) that is

TABLE 2. Comparisons of earthquake code requirements for residential construction

Requirement	1985 UBC	1985 NBC Part 9
Wall bracing (in-plane)	2517(g)3	Not covered except for post and beam construction Part 4 via 9.24.1.5
Cripple stud foundation walls	2517(g)4	Part 4 via 9.15.1.5
Anchorage to foundations	2907(f)	9.23.6
Beam splice ties over supports	2517(c)	Not covered
Lateral support of masonry walls	2407(e)	9.20.10 and 11
Anchorage of masonry veneer	3006	9.20.9.9
Reinforcing of masonry	2407(h)4B	9.20.17
Anchorage and reinforcing of masonry chimneys	3704(c)	Not covered
Stability of masonry parapets	2312 Table 23-J	9.20.6.7

likely to produce a higher ground acceleration is not considered here.

Damage estimates

Major damage in the Greater Vancouver area from a design-level earthquake can be expected to be greater than what was experienced in the San Francisco area for several reasons: (*i*) the design level for earthquake resistance (NBC Part 4) is lower than in the San Francisco area, (*ii*) the seismic requirements for non-engineered construction (NBC Part 9) are less stringent, (*iii*) the extent of soft soils in populated areas is larger than in the San Francisco Bay area, and (*iv*) the recorded ground motions in the most densely populated areas, San Francisco and Oakland, were from one half to two thirds the design earthquake for Vancouver.

Foundation failures and liquefaction in fill areas during the Loma Prieta earthquake were encountered in areas that had a peak ground acceleration on firm ground as low as $0.10g$. This confirms the potential problems with soft soils in the Vancouver area, including the Fraser Delta, the False Creek area, and other such deposits and man-made fills.

As a result of the experience with the Loma Prieta earthquake and an evaluation of some previous earthquakes on the Pacific coast, as well as considerable judgement, an estimate of major damage effects for various types of buildings and services in the Vancouver area was arrived at as follows:

Single-family housing and low-rise residential and office buildings

- Superficial damage was initiated at $0.10g$ in the Loma Prieta earthquake (Table 1).
- Extensive damage was done to inadequately braced or geometrically problematic buildings at $0.20g-0.40g$.

It is our judgement that at $0.20g$ some buildings would be damaged where, at least initially, occupancy is considered unsafe. Some of these failures would be due to liquefaction of sand. A loss ratio for these types of buildings is estimated in the range of $2-5\%$.

Unreinforced masonry buildings

- In the downtown Oakland area with a ground acceleration at $0.20g-0.30g$, most unreinforced masonry multistorey buildings were damaged so that they had to be evacuated.
- In Watsonville and Santa Cruz, at a ground acceleration between $0.30g$ and $0.40g$, more than 60% of the unreinforced masonry buildings were seriously damaged or collapsed.

We estimate that at $0.20g$, from 20% to 50% of unreinforced masonry buildings would be seriously damaged. Much of the damage would occur in masonry with deteriorated mortar or bricks.

High-rise residential buildings

- Most multistorey buildings sustained heavy damage in the San Fernando 1971 earthquake at $0.20g-0.30g$ ground acceleration.
- In the Loma Prieta earthquake most buildings of intermediate height ($5-10$ storeys) performed well in the areas of around $0.30g$ peak ground acceleration in the Palo Alto and surrounding area.
- High-rise office buildings of newer construction in San Francisco experienced no substantial structural problems at $0.10g-0.15g$ ground acceleration, well below the seismic design level for that city.

In the Vancouver area, many high-rise residential buildings have been designed to lower levels of resistance than in the San Francisco area at a time when the material standards were less stringent than what is considered appropriate now. Consequently, it is estimated that $5-10\%$ of these buildings would experience significant damage at $0.20g$ ground acceleration. Some of these that are located on liquefiable sands would be adversely affected by foundation problems.

Schools and hospitals

- These performed well in the San Francisco area at ground motions ranging from $0.10g$ to $0.40g$, although temporary power disruptions were experienced in the Watsonville Hospital.
- In the San Fernando earthquake of 1971 many schools were seriously affected at ground accelerations from $0.20g$ to $0.30g$.

With the ''Field Act of 1935'' governing seismic resistant school construction in California and the stringent seismic requirements for hospitals in effect, these structures have shown good performance. However, the same requirements are not in effect in British Columbia and thus the structures are judged to be more vulnerable, with an estimated loss ratio of $10-30\%$ for a $0.20g$ ground acceleration. For strengthened structures, the losses are judged to be marginal at a $2-5\%$ loss ratio.

Services

- The entire San Francisco Bay area was without power for one night as a result of a substation failure, most power was being restored after a few days, however.
- Water was cut off in areas of soft soil deposits.
- Transportation routes were seriously disrupted by the failure of a few bridges and freeways.
- Some minor failures of sewage treatment plants were reported in areas of $0.20g-0.40g$ ground acceleration.

We estimate that a similar situation is likely to exist in the

Vancouver area and place the damage ratio from 5% to 10% for unavailability of the service. In the areas that are served exclusively by one substation or one water trunk line and these failed, the loss ratios would reach 100%, however.

Harbour facilities and airport runways

Because of the proximity of soft soil deposits of these structures they are subject to liquefaction and sliding, given the appropriate conditions. At 0.20*g* ground acceleration it is estimated that from 5% to 20% of harbour structures and runways would be seriously affected and not be available for use. Airport structures, on the other hand, are expected to be only marginally affected, at a loss rtaio of 2−5%, since many of them are newer structures built with counter-measures for liquefaction such as soil compaction and piles.

The results are summarized in Table 3 in terms of estimated regional loss ratios. These estimates apply to large sections of the Greater Vancouver area that receive ground shaking at or near the design earthquake 0.20*g*, but are not meant to reflect a numerical loss ratio of all existing houses, since not the entire metropolitan area will experience the same level of shaking. A full assessment of total damage would require a detailed seismological investigation, microzoning of local hazards of ground shaking, a detailed inventory of building stock, and a more detailed assessment of expected seismic behaviour of these structures. The present study is a first step in the direction of such a detailed assessment.

TABLE 3. Summary of estimated major structural damage from design earthquake over major regions of the Greater Vancouver area

Type of building or service	Estimated loss ratio (%)
Single-family houses of wood frame construction	2−5
Unreinforced masonry	20−50
Low- and medium-rise residential and office	2−5
High-rise residential	5−10
Schools and hospitals	
Prior to 1940 old construction (not strengthened)	10−30
Newer construction and strengthened old construction	2−5
Gas and water supply, sewers	5−10
Electricity	5−10
Communication systems	5−10
Transportation routes (bridges)	5−10
Harbour facilities	5−20
Airport structures	2−5
Airport runways	5−20

Summary and conclusions

The Loma Prieta earthquake that hit the San Francisco area on October 17, 1989, caused a total of $10 billion (U.S.) damage and over 60 deaths. Buildings built in the last 20−30 years and located on firm ground performed well, while some older houses and those located on soft deposits suffered major damage and often collapsed. Soft soil deposits have again demonstrated their potential for amplification of shaking and loss of bearing capacity due to liquefaction, with subsequent risk to structural integrity and safety to occupants. Other structures that suffered major damage were older wood buildings with inadequate lateral resistance at the ground level, and unreinforced masonry buildings and elevated concrete highways.

The conditions found in the San Francisco area were extrapolated to what might be expected in a "design earthquake" in the Greater Vancouver area. The predictions are based on considerations of the seismic requirements for engineered and non-engineered construction, the extent of soft soil deposits, and the somewhat lower peak seismic ground motions in the most populated area of the Bay area, San Francisco, and Oakland, as compared to the design earthquake for Vancouver.

A comparison between the 1985 Uniform Building Code and the National Building Code of Canada shows that the requirements for non-engineered buildings, Part 9, have potential shortcomings concerning lateral bracing, beam splice ties over supports, and anchorage and reinforcing of masonry chimneys.

The experience from the San Francisco area indicates that upgraded buildings performed well. Upgrading of vulnerable construction should continue to be pursued in the Vancouver area and other areas with significant seismic potential. For these cases, alternative measures are needed to the NBC design criteria that are applicable to new construction.

The high level of emergency preparedness in the affected San Francisco area provided for effective rescue operations, relief and care for evacuated people, and a return to near-normal operating conditions of community within a few days of the earthquake.

The main factors causing damage in a design-level earthquake in the Vancouver area are (*i*) soft soil deposits; (*ii*) old unreinforced masonry and concrete construction; (*iii*) buildings that were designed to previous codes and standards that are currently not considered adequate; (*iv*) design weaknesses not covered by codes and standards, such as minimum bracing in NBC Part 9; and (*v*) workmanship defects that seem to occur on typical job sites.

It is strongly recommended that these factors be addressed, and that existing construction be evaluated and retrofitted where necessary. This includes the continued examination and revision, where necessary, of applicable building regulations.

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