Preliminary report of structural damage from the Loma Prieta (San Francisco) earthquake of 1989 and pertinence to Canadian structural engineering practice

MICHEL BRUNEAU

Department of Civil Engineering, University of Ottawa, 161 Louis Pasteur, Ottawa, Ont., Canada K1N 6N5

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The Richter magnitude 7.1 October 17, 1989 Loma Prieta (San Francisco) earthquake is the largest to occur near a major North American urban center since the historical 1906 San Francisco magnitude 8.3 earthquake. As earthquakes of at least similar strength are expected to occur in most of eastern and western Canada, and since the amount of structural damage that occurred is considerable, the study of the effects of this earthquake is of particular significance to Canada. This paper reports on the major structures and types of structures that were most heavily damaged by this earthquake, and presents preliminary findings as to the causes of failures or collapses. The pertinence of this earthquake is reviewed in a Canadian perspective.

Key words: earthquake, structures, damage, failure, collapse, buildings, bridges, heritage buildings, emergency preparedness.

Le tremblement de terre Loma Prieta (San Francisco) du 17 octobre 1989, mesurant 7,1 à l'échelle Richter, est le plus puissant séisme à se produire à proximité d'un important centre urbain nord-américain depuis celui de San Francisco en 1906, estimé à 8,3 à l'échelle Richter. Puisque des séismes de puissance égale ou supérieure à celui de Loma Prieta sont attendus dans la majeure partie de l'est et de l'ouest canadien, et vu le dommage considérable observé suite au récent tremblement de terre, l'étude des conséquences de ce séisme est d'une importance capitale pour le Canada. Cet article rapporte sur l'état des structures et types de structures les plus sévèrement endommagées, et présente des conclusions préliminaires quant aux causes de leur ruine totale ou partielle. La pertinence de ce séisme est discutée dans le contexte canadien.

Mots clés : tremblement de terre, structures, dommages, ruine partielle, ruine totale, édifices, ponts, édifices du patrimoine, protection civile.

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1. Introduction

On October 17, 1989, at 17:04 Pacific standard time, a Richter magnitude 7.1 earthquake occurred along the San Andreas fault near mount Loma Prieta in the Santa Cruz mountains and was strongly felt in the San Francisco–Oakland–San Jose Bay Area. Although the epicenter of the earthquake was located more than 80 km south of the San Francisco–Oakland axis, this earthquake will likely be remembered as the San Francisco earthquake of 1989, mainly because of the larger population density of the Bay Area and the dramatic collapses and failures that occurred there, not to mention the extraordinary national and international media presence in San Francisco for the coverage of game 3 of the baseball World Series, which was interrupted and cancelled due to the earthquake.

This earthquake is the strongest and most devastating to hit northern California since the 1906 Richter magnitude 8.3 San Francisco earthquake. It may nevertheless be the most costly disaster in North American history as losses are preliminarily estimated to range between 7 and 10 billion U.S. dollars; early estimates also indicate that up to 105 000 houses and 320 apartment buildings suffered some form of damage, the operation of 1345 businesses has been affected, and over 13 000 persons have been displaced by this earthquake. The recent seismic event will provide much needed data on the impact of a large earthquake near modern North American urban centers of high population density. It is noteworthy that a similar earthquake had previously occurred in the Santa Cruz region on October 8, 1865; the area was then scarcely populated, thus damage was not significant and no deaths were reported.

The occurrence of severe damage to many engineered structures 80 km away from a Richter magnitude 7.1 earthquake (having only about 1/60 of the energy of the San Francisco 1906 magnitude 8.3 historical event) is a major cause of concern. While this small-scale disaster will provide Californians with a golden opportunity to reassess their overall earthquake preparedness, it is also of uppermost importance for Canadians to study the resulting structural damage as earthquakes of at least magnitude 7.0 are expected to occur near many cities in eastern and western Canada, and because a large number of existing structures in these regions, never designed to withstand such high level of lateral forces, are exposed to this risk.

Shortly after the occurrence of the main shock, the author undertook a 6-day intensive structural damage reconnaissance visit to review the nature of structural failures that occurred and assess the significance of this damage in a Canadian perspective. This paper is intended to provide a brief description of the seismic characteristics of this event, a report on the various types of damage and major modes of structural engineering failures that occurred, and recommendations on how the collected data can be used to improve Canadian earthquake preparedness. It should be understood that the author's conclusions are preliminary only.

Future research on the collected data will provide greater insight in the seismic structural behaviour of existing structures, as well as more definitive conclusions on the causes of failures. The current concentration of earthquake specialists in the area directly affected will undoubtedly propel this research effort.

NOTE: Written discussion of this paper is welcomed and will be received by the editor until August 31, 1990 (address inside front cover).

2. Seismicity considerations

The San Andreas fault, a right-lateral strike-slip fault, is estimated to have ruptured for a length of 40 km, with only small evidence of surface rupture reported at the time of this writing. Aftershocks have been scattered along the fault segment from the Lexington Reservoir to San Juan Bautista. Two Richter magnitude 5.4 earthquakes had previously occurred in this same area in June 1988 and August 1989; in retrospect, these may have been foreshocks of the larger Richter magnitude 7.1 October event. It should be realized that although a 30% probability of occurrence in 30 years of a Richter magnitude 7.0 had been assigned along this segment of the San Andreas fault (USGS 1988), current state-of-the-art seismology does not allow shorter-term forecasting.

While long-term predictions provide little warning of imminent danger, they should provide a sufficient framework for the formulation of earthquake preparedness policies and structural engineering retrofitting strategies. Unfortunately, while such probabilities have been formulated in Canada, they have not been met with a sense of urgency. For example, unreinforced masonry hospitals and other postdisaster buildings remain a common sight both in western and in eastern Canada; the larger level of awareness of the west coast should nonetheless be acknowledged and has generally led to recent increases in preparedness activities.

The October 17, 1989 earthquake is only the largest California earthquake since the July 20, 1952 Richter magnitude 7.7 Kern County earthquake, but is certainly the strongest and most devastating to hit northern California since the 1906 Richter magnitude 8.3 earthquake, which had cost 700 lives and caused property losses of 400 millions U.S. dollars, although approximately 80% of those losses are attributed to the ensuing fire that raged in the city for 3 days following the earthquake (Bolt 1978). An estimated 430 km of the San Andreas fault then ruptured, with horizontal displacements of 4.5–6.0 m (15–20 ft) (Bolt 1978). The relationship between earthquake magnitude and released energy (Rosenblueth 1980) reveals that the recent earthquake had only about $1/60$ of the energy of the 1906 historical event. Clearly, significantly more damaging earthquakes can potentially occur in the Bay Area.

Further, the occurrence of the October 17, 1989 earthquake near Santa Cruz, while significant, has relieved little of the energy accumulated along the San Andreas fault elsewhere, and probabilities of occurrence of a large earthquake of magnitude 7.0 or more in the San Francisco area remains practically unchanged, namely at 30% probabilities of occurrence over the next 30 years (USGS 1988). Also unaffected remain the probabilities of occurrence of similar earthquakes along the northern and southern parts of the Hayward fault on the East Bay, each currently estimated at 20% in 30 years (USGS 1988).

A large number of strong motion instruments have recorded the latest event. The Office of Strong Motion Studies of the California Division of Mines and Geology (CSMIP 1989) estimates that over 90 of their seismograph stations have recorded the earthquake. Table 1 summarizes some of the horizontal and vertical peak ground accelerations (PGA) recorded by a number of stations, geographically identified in Fig. 1, as of October 19, 1989. While a maximum horizontal PGA of $0.65g$ has been recorded by the Corralitos station, at 15 km from the epicenter, the PGA has rapidly decreased to $0.11g$ 40 km away in some instances. The time history of the base acceleration of the Corralitos station (CSMIP station 57007) is presented in Fig. 2. Strong motion can be seen to last approximately 10 s, with some relatively long frequency oscillations. In a separate communication, the University of California, Berkeley, indicated that its seismograph station recorded a PGA of $0.09g$.

3. Damage survey

Generally, structural damage has not been uniformly widespread, but has rather been localized to areas where structures were built on soft soils. A comparison of locations where severe structural damage occurred in San Francisco due to both the 1906 and 1989 earthquakes (Fig. 3) emphasizes this observation; in both cases, damage was concentrated where land has been reclaimed from the bay. Consequently, most of the Bay Area survived the earthquake unaffected.

Following is an overview of the damage observed in the visited areas: the collapsed portion of Highway I-880 in west Oakland; the San Francisco – Oakland Bay Bridge; the Marina District, Embarcadero Freeway, downtown and south of Market street districts in San Francisco; Highway I-280 in south San Francisco; downtown Oakland; and the downtowns of Santa Cruz, Watsonville, Hollister, and Los Gatos. These locations are highlighted in Figs. 1 and 3. The author has limited his survey to buildings, bridges, and residential structures, but it should not be inferred that these were the only structures affected by the earthquake. In fact, the numerous water and gas lines breaks, as well as telecommunication overloads, testify that lifelines were also seriously affected.

3.1. Highway I-880

An approximately 2 km stretch of the elevated highway I-880, crossing west Oakland, has completely collapsed (Fig. 4). This double-decker highway carried 4 lanes of traffic in each direction. Typically, after failure of its supports, the upper deck fell on the lower deck, trapping a number of vehicles between the decks and killing a large number of people.

There is not sufficient evidence at this time to determine if all affected spans collapsed simultaneously or if a "domino" effect was initiated by the failure of a single span. Nevertheless, all spans between the points where the freeway changes from a double to a single deck highway partially or totally collapsed.

Alongside the highway is a large number of industrial and warehousing facilities constructed of unreinforced masonry and tilt-up construction which appeared to have remained uncracked. This indicates that ground accelerations at the base of this structure could have been relatively low; sufficiently low to leave unaffected a type of construction usually seen to have unsatisfactory behaviour during an earthquake. It is noteworthy that many regions of eastern and western Canada could be excited by ground accelerations of a magnitude equal or larger than those that led to the collapse of I-880. It should nevertheless be also mentioned that this elevated highway is located on landfill material; it is therefore possible that the soil layers have not only amplified the base-rock excitations but also "filtered" its frequency content with resulting surface ground motions more susceptible to detrimentally affect long-period structures than short-period ones. Detailed geotechnical studies will provide further insight in the probable ground surface excitations that have occurred at that location.

This two-tiered elevated segment of Interstate I-880 was completed in 1957. Nonetheless, the dramatic structural

TABLE 1. Peak ground accelerations reported by CSMIP* stations

Station name	Distance from epicenter (km)	Peak ground acceleration (× g)	
		Horizontal	Vertical
Corralitos	5	0.64	0.47
Watsonville — base of Telephone Building	11	0.39	0.66
Capitola	14	0.54	0.60
Gilroy array station 1 (rock)	21	0.50	0.22
Gilroy array station 2	22	0.37	0.31
Gilroy array station 3	24	0.55	0.38
San Juan Bautista — base of 101 overpass	25	0.15	0.10
San Jose — base of Santa Clara Building	40	0.11	0.10
Hollister — Warehouse FF	40	0.38	0.20
Foster City — Redwood Shores	70	0.29	0.11
Hayward — Muir School	77	0.18	0.10
San Francisco International Airport	87	0.33	0.05
Los Banos	88	0.05	0.01
South San Francisco — base of 4-storey hospital	93	0.15	0.08

*California Strong Motion Instrumentation Program, Office of Strong Motion Studies, Division of Mines and Geology, Department of Conservation, State of California.

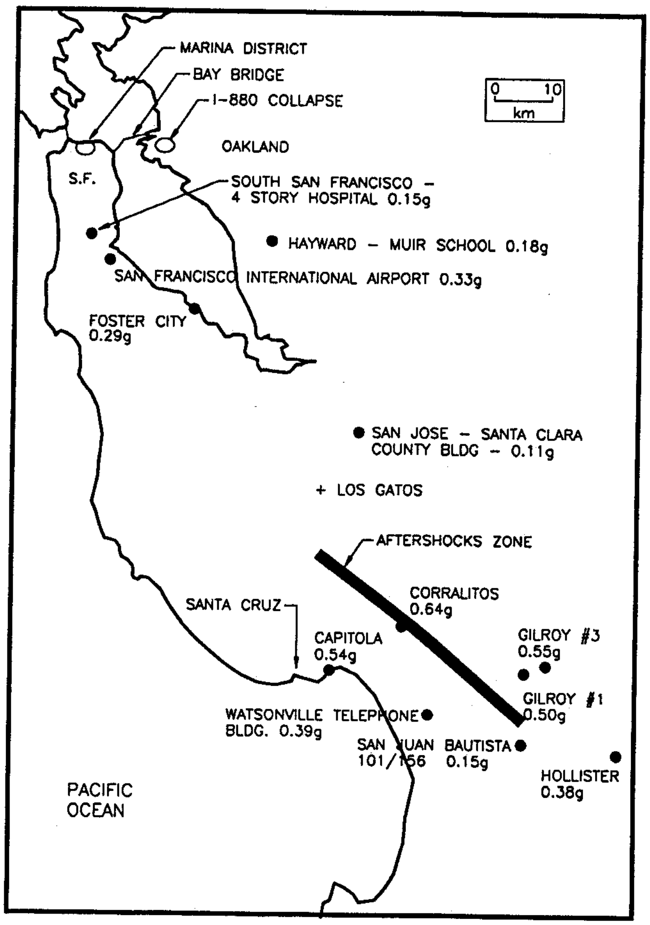

FIG. 1. Location of seismograph stations, damage areas, and aftershocks zone (adapted from CSMIP 1989).

failure was not anticipated by the State of California's Department of Transportation (CALTRANS). The resulting failure mode of this highway was unlike anything previously seen.

It is premature to describe with certainty the exact failure mode and collapse mechanism of this highway; a comprehensive investigation is currently underway and results are anticipated to be of most interest to the bridge engineering community. Nevertheless, a number of particular features of this structure are of uppermost interest and should be described. Also, a preliminary assessment of the failure mechanism has been conducted by engineers at the University of California, Berkeley, and results of their initial findings are presented herein.

3.1.1. *Features of I-880 elevated highway*

The American Association of State Highway Officials (AASHO) design standard edition in effect at the time of the original design did not provide seismic loads for which bridges had to be designed (AASHO 1957). Nevertheless, it appears that in spite of this "missing load case," Californian bridge engineers in those days were designing bridges for some nominal level of earthquake excitation. The exact level and nature of the seismic forces considered in the original design of this structure will only be known with certainty following CALTRANS' detailed investigation. However, the design force may be irrelevent. Looking at member sizes and reinforcement of the existing collapsed structure, it is clearly the conceptual design of the elevated highway which has had a most negative impact on its ultimate behavior, not so much the level of lateral forces for which it was designed.

Figure 5 presents schematically the highway cross section. In most instances, columns supporting the upper deck were hinged such as to make the system statically determinate. Typically, only the base of one column was not hinged, and

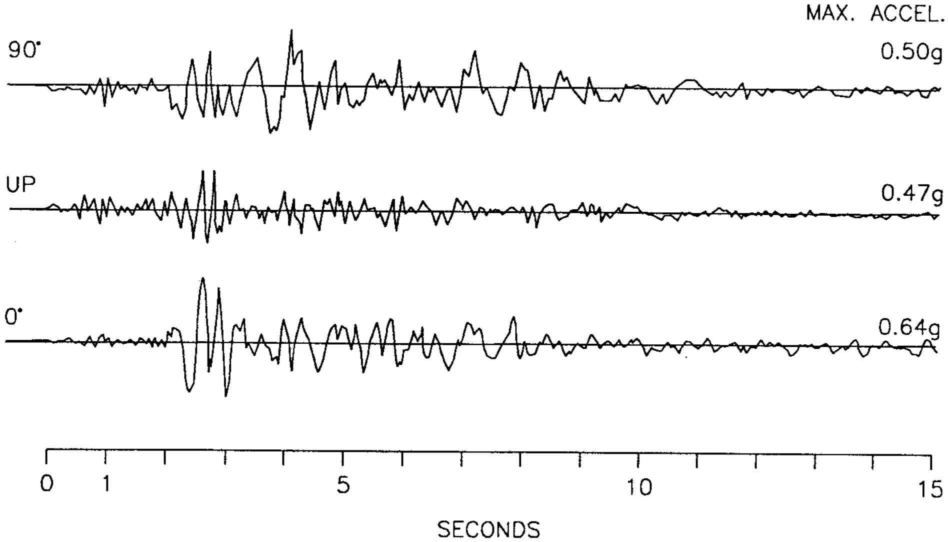

FIG. 2. Ground accelerations seismograph records — Corralitos, CSMIP Station 57007, Record 57007-S4809-89291.00 (adapted from CSMIP 1989).

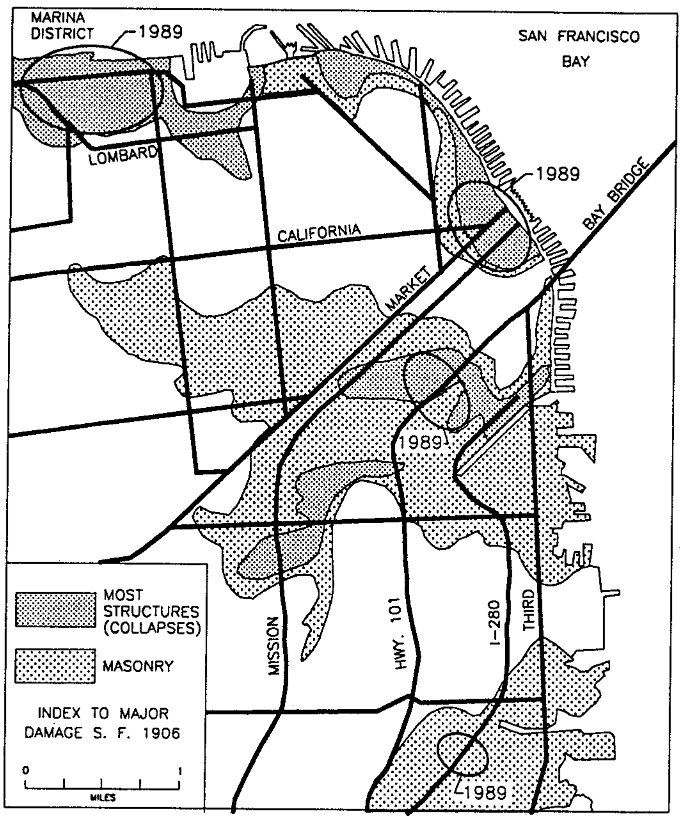

FIG. 3. Location of structural damage in San Francisco from the 1906 (adapted from Iacopi 1981) and 1989 earthquakes.

this "continuous" column was not regularly constrained to be on the same side of the highway. The original drawings will be required to determine the actual hinges layout. Hinges were accomplished by the insertion of a felt-type material effectively reducing the concrete area of the column from a 38 × 48 in. (97 × 122 cm) heavily reinforced cross section to a 36 × 22 in. (91 × 56 cm) core with only 4 × #11 bars continuous for approximately 2 ft (0.60 m) beyond the hinge into the lower column (Fig. 6). The original hinge design was intended to make the higher level of the structure statically determinate, thus simplifying the evaluation of thermal and traffic overloads stresses in the structure, as well as stresses due to a prestressing scheme across the width of the upper deck that could have been implemented had it been judged necessary to connect additional access ramps to the upper level of the structure after completion of the highway.

3.1.2. Preliminary assessment of the failure mechanism of the I-880 elevated highway

There has been extensive speculation as to the cause of this failure. The absence of ductile shear reinforcement in the column, an item intensely debated in the days following failure, was indeed a design deficiency which would have been a weak link in normal circumstances, but a weaker link was in fact present in this structure as revealed in a "Preliminary Briefing on the Effects of the October 17, 1989 Earthquake" by engineers from the University of California, Berkeley. As shown in Fig. 7, the absence of joint steel reinforcement to produce an effective confinement and continuity between the horizontal reinforcing of the beams and the vertical reinforcing of the columns, coupled with the discontinuous nature of column reinforcement at the hinges, have left the joints very vulnerable to brittle shear failure. Based on his personal observations of the damage, the author supports this latest theory. Diagonal cracking observed in the joints of parts of the elevated Embarcadaro freeway and Interstate I-280 in San Francisco, of similar design and constructed in the same era, confirms the joint's exceptional vulnerability for this type of two-tiered highway.

Typical joint failures of hinged and continuous columns of the highway I-880 are shown in Fig. 8, where the failure surfaces are clearly seen to be above the bent of the horizontal

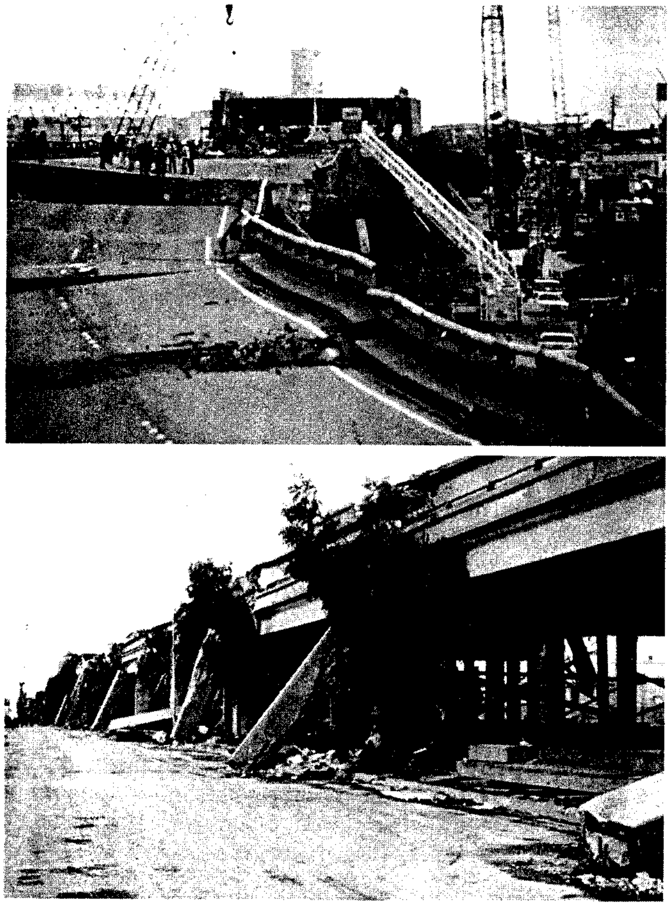

FIG. 4. I-880 highway in west Oakland: (*a*) looking longitudinally south from north end; (*b*) looking at elevation from west side.

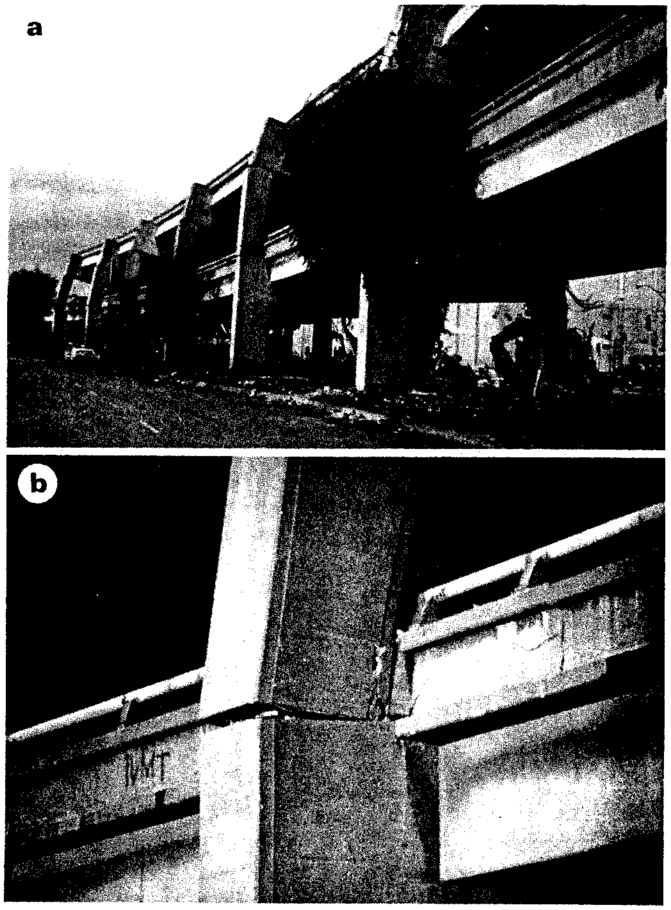

FIG. 6. (*a*) General view of typical hinged upper column on west side; (*b*) close-up view of same.

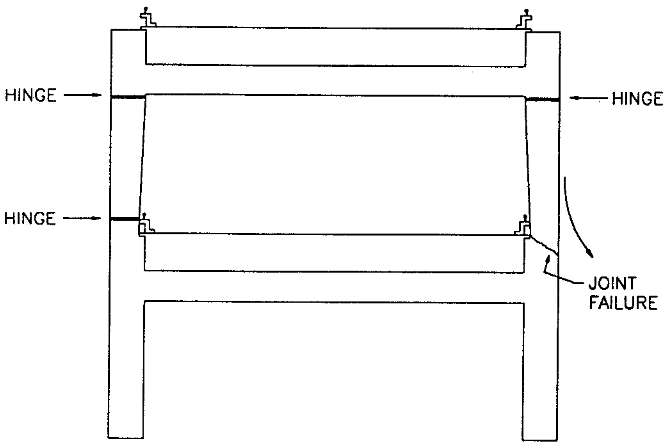

FIG. 5. I-880 highway schematic cross section, structural concept, and location of initial failure.

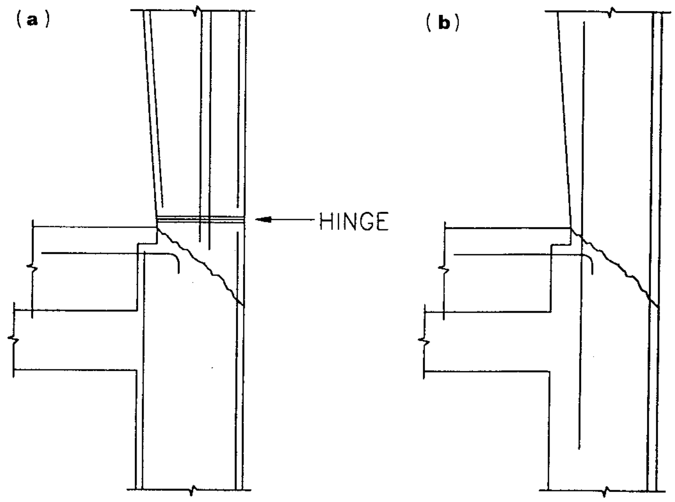

FIG. 7. Schematic illustration of joint failure at (*a*) hinged base of upper column and (*b*) continuous base of upper column.

reinforcement, and to intersect no stirrups. It should nevertheless be noted that if a single frame would have been present with both columns hinged top and bottom, a mechanism would have been created with no theoretical lateral load resistance, but no such frame has been identified at this time. Finally, a resonance phenomenon should not be ruled out, as the exact influence of the soft underlying soils on the ground motion characteristics are also uncertain in this particular case.

Preliminary estimates of the internal stresses acting in the joints could be provided, but simple analysis methods may overlook severe stress concentrations in the joint, and consequently overestimate the joint capacity. This may be of large significance in this particular case where little, if any, reinforcement contributed to the shear resistance of the joint.

FIG. 8. Typical joint failure at (*a*) base of hinged upper column and (*b*) base of continuous upper column.

Dynamic analyses, finite element analyses, and potentially fracture mechanics analyses would provide further insight into the ultimate failure mechanism. Without the benefit of such analyses, the author wishes to limit the presentation of quantitative data to self-weight considerations: measurements and preliminary calculations by the author indicate that the upper deck would have weighed 190 kN/m (13 kips/ft), for an unfactored dead load of 2670 kN (600 kips) at each column of this upper level.

3.2. San Francisco – Oakland Bay Bridge

The Bay Bridge, opened to traffic November 12, 1936, consists of a suspension section connecting San Francisco to Yerba Buena Island, and a cantilever section from that island to Oakland. Trains were originally carried along the southern half of the lower deck, and lighter traffic on the upper deck, which explains the unusual variable depth/spacing girder configuration that can be observed (Fig. 9). The strengthening solutions implemented during conversion to traffic only, while interesting, have not modified the fundamental design concept in the damaged area. The only damage to this very long span occurred above pier E-9. This structural steel pier is of a four-legged very rigid braced design, schematically reproduced in Fig. 10. The truss bridge sections east and west of pier E-9 were rigidly connected to the east and west legs of the pier respectively. The roadways built across this pier were simply supported. The original design assumption was that each side of the pier would not move relative to each other. Yet, after the earthquake, the residual displacement of the bridge section east of this pier was 13 cm (5 in.) east (longitudinally) and 2.5 cm (1 in.) north (laterally), and the twenty 2.5 cm (1 in.) diameter bolts that connected it to the pier were sheared off. It is estimated that the bridge might have moved 18 cm (7 in.) eastward during the earthquake. The simply supported spans across pier E-9 were connected to the east part of the bridge that broke free, and rested on seat connections with web alignment plates, totally free to move, on the west side (Fig. 11). The unattached side of the spans simply slid off their supports to produce the observed collapse.

It is noteworthy that movement restrainers, in the form of steel buttress beams installed at the east end of the bridge to limit its lateral movement and which have served their function during the earthquake, have not been sufficient to prevent the aforementioned failure.

Soil liquefaction has also been extensive throughout the landfill approaches to the bridge. Nevertheless, the Toll Building survived the earthquake well, despite some minor damage, and was used as a CALTRANS Command Post following the earthquake. This is significant as it demonstrates that some old structures have been successfully engineered to survive being located on severely seismically hazardous soils. Further studies of this building and its foundations would be most interesting.

3.3. Marina district

A large number of residential dwellings in the Marina district collapsed during the earthquake. A major fire was also ignited by a gas leak. Water mains were broken and a conflagration may have only been avoided due to the proximity of the Marina district to the Bay where a fire-boat provided the pumping pressure needed to fight the blaze. The author has not been able to verify at this time whether the doubled water main and underground reservoir backup system, in place in most of San Francisco for fire-fighting purposes, extended to the Marina district.

Again, very soft soils of reclaimed land underlied the most severely damaged structures. While this has likely produced an amplification of the otherwise moderate ground motion, and a large number of instances of localized soil liquefaction, the collapses should have been predictable by simple observation of the typical structural systems used.

As seen in Fig. 12 of typical residential buildings which nearly collapsed, the first story of such structures is mainly open to be used for parking purposes. A softer story is there-

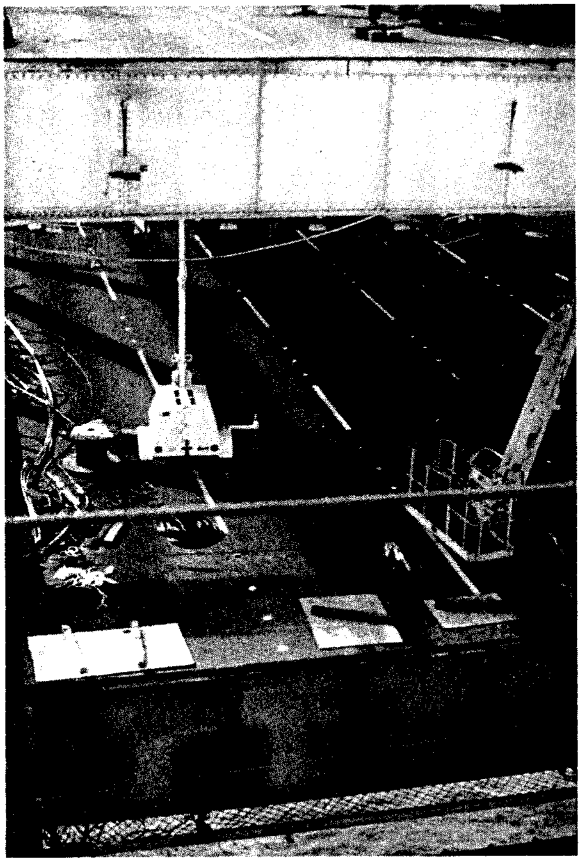

FIG. 9. West side supports of span across pier E-9 of San Francisco–Oakland Bay Bridge.

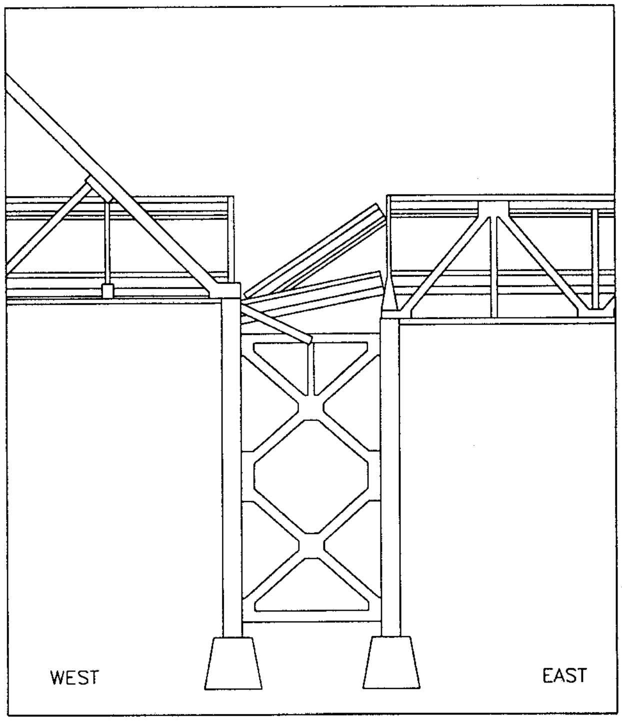

FIG. 10. Schematic representation of pier E-9 (Bay Bridge).

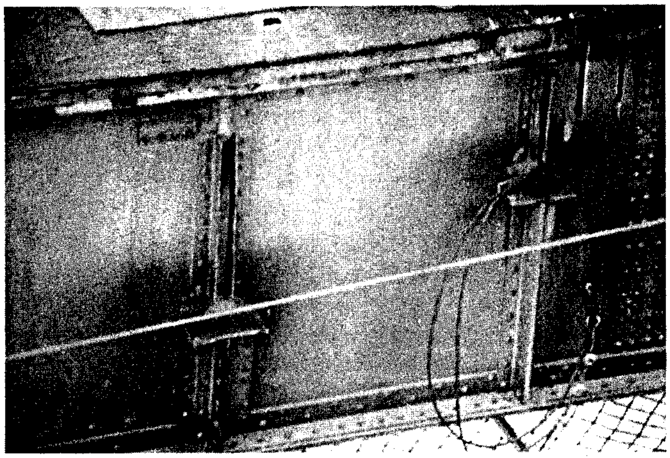

FIG. 11. Typical beam seat and web alignment plates (Bay Bridge).

fore created, i.e., a story where a relatively small number of partitions is available to resist the seismic excitation, when compared to the upper stories which are subdivided by a large number of wall partitions. Consequently, all the energy dissipation (i.e., structural damage) will be constrained to take place in the first story. It is noteworthy that most modern structural engineering codes are not rigorously designed to prevent this very undesirable behavior. Further, in most damaged buildings, not only were there few structural elements on the first story effective in resisting lateral forces, but those remaining elements were often walls of latticed wood construction, not believed very good in transferring the seismically induced shear forces. Soft stories have been recognized to be a problem in engineered structures since the Richter magnitude 6.6 1971 San Fernando Valley earthquake (Bertero 1979); this is the first time it has been observed on residential structures to such a large scale.

While this conceptual construction type is widespread in California, it is also extremely popular in Canada, notably for large apartment complexes in British Columbia, where it is ideal to protect cars from the rain. It is also likely to become increasingly widespread as a parking solution as city cores get repopulated in answer to deteriorating commuting conditions. An evaluation of these structural systems, where present in Canada, would be warranted in order to assess their performance under various levels of earthquake excitation typical of eastern and western Canada's seismicity. However, this may effectively be difficult to achieve as these targeted structures are often nonengineered.

3.4. *Unreinforced masonry buildings*

Not surprisingly, unreinforced masonry buildings were the most severely hit during this earthquake. Damage to masonry structures was observed in Oakland and San Francisco, and most dramatically in Santa Cruz, Watsonville, Hollister, and Los Gatos.

Most well-known types of unreinforced masonry building failures were observed, including severe diagonal cracking in columns between windows (Fig. 13), loss of masonry walls improperly tied to the rest of the building (Fig. 14), roof joists or beams slipping off their supporting wall (Fig. 15), fallen parapets, cornices, exterior cladding/glazing or veneers, or decorative elements, and, of course, failure of the building as a whole due to insufficient lateral load resistance. Pounding of adjacent buildings was also apparent in many instances.

It is paradoxical that many North American towns and cities

FIG. 12. Typical residential structures with "soft-story" problem (Marina district, San Francisco).

invest large sums to renovate and make attractive their main street in order to enhance and preserve their heritage flavor (and simultaneously attract tourism) while not addressing the sesimic adequacy of these downtown districts, predominantly of unreinforced masonry construction. In Santa Cruz, the Pacific Garden Mall was such a district. This open-air shopping complex of more than 30 buildings was a total loss, with all structures sufficiently damaged to be declared hazardous and demolished. The loss of The Cooper House (Fig. 16), a turn-of-the-century heritage building that had become a landmark to Santa Cruz, is such an example. It would be of uppermost importance to study this particular structure in greater details, to determine analytically its vulnerability threshold, compare its construction to that of similar Canadian heritage structures, and use it as a yardstick of the seismic vulnerability of heritage structures in Canada. The same could be done for a number of other heritage structures such as the 100 Front Street building (1893) in Watsonville, or the Palomar Inn in Santa Cruz.

In Hollister, a few buildings, apparently in good structural condition and adjacent to others that had suffered significant damage, had apparently been seismically retrofitted shortly before the earthquake. These may provide valuable data with regard to the performance of actual seismically retrofitted unreinforced masonry structures.

3.5. Single-family residential buildings

Numerous single-family residential dwellings have been

FIG. 13. Typical short column shear failure of masonry structure (Oakland).

severely damaged in regions nearer the epicenter. While it is generally accepted that these structures of wood-frame construction, due to their large structural redundancy and energy dissipation capabilities, can survive earthquakes very well (except for toppled brick chimneys or when irregular layout and configuration are used), this is assuming that such houses are properly attached to their foundation. Improper foundations or connection to foundations are responsible for most of the residential single-family dwellings damaged by the October 17 event.

As shown in Fig. 17, a widely used older residential construction practice consisted of using small "pony wall," typically 46 cm (18 in.) tall, in order to raise the first floor above concrete foundations extending only a few centimeters beyond the surface; pony walls were made of wood construction, and in most cases poorly connected to the foundation. Rigid-body rotation of these walls displaced the houses laterally under seismic excitation, a deficiency not unlike that of a soft-story concept. Newer residential construction has fared generally very well.

4. Pertinence to Canadian structural engineering practice

Most of the observed damage from the October 17, 1989 earthquake is directly pertinent to Canadian seismic-resistant design and earthquake preparedness, as many eastern and western Canadians are within or near geological regions deemed capable of generating Richter magnitude 7.0 or greater earthquakes (Basham *et al.* 1982).

The major importance of ground excitation amplification attributed to soft soils is again emphasized by this earthquake. The cities of Quebec City and Richmond, B.C. are only two examples of cities for which such concerns should be of prime importance. Not surprisingly, the maximum foundation factor F (of the National Building Code of Canada (NBCC)) is expected to be increased to 2.0, instead of 1.5, a more appropriate representation of the amplifications expected to occur on the softer soils.

Furthermore, damage observed to older bridges and buildings far from the epicenter, thus at low peak ground accelerations, is of a direct pertinence to Canadian structural engineering practice, especially since many existing structures in Canada have never been designed to resist earthquakes. The deficient seismic strength or ductility of many older existing

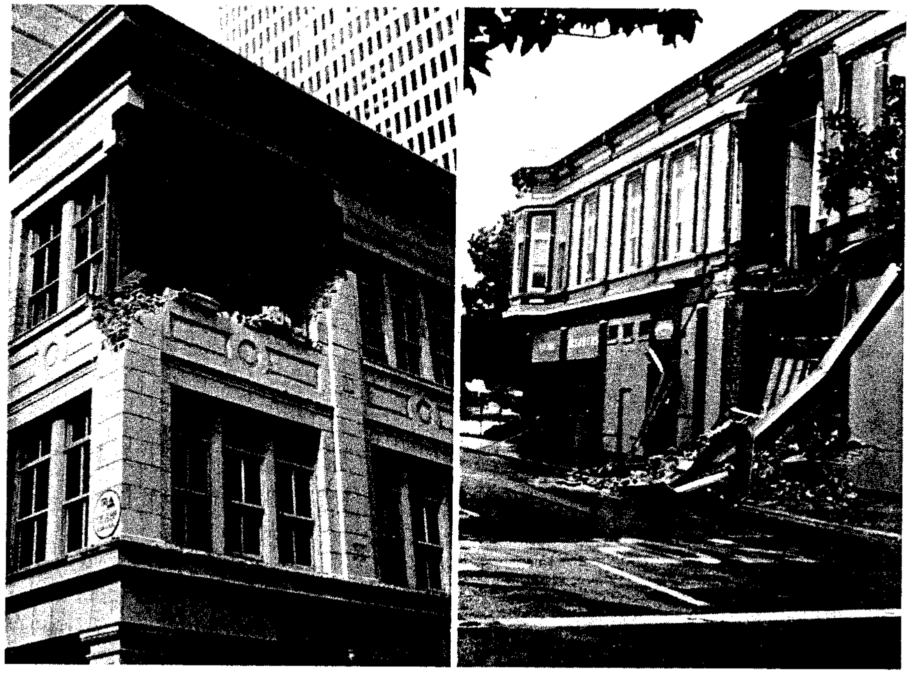

FIG. 14. Typical loss of masonry walls (San Francisco and Santa Cruz).

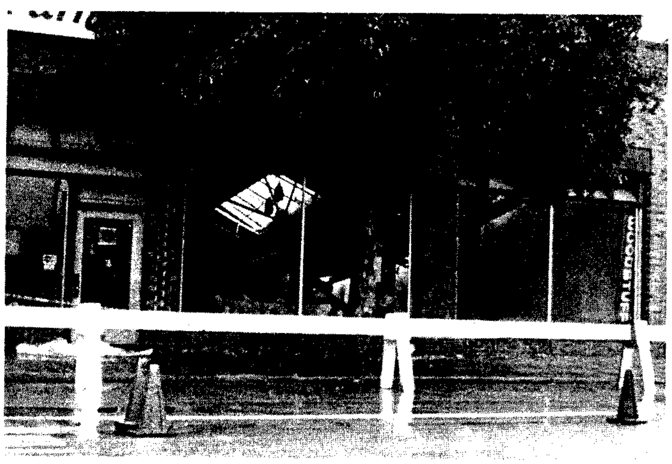

FIG. 15. Typical failure due to slip of roof joists off their wall supports (Santa Cruz).

FIG. 16. The Cooper House (Santa Cruz).

structures is a particularly acute problem in Canada as the first effective seismic requirements were implemented in the NBCC in 1953, and adopted by local by-laws only as late as 1967 in some cases. In many instances, these older structures are of unreinforced masonry, the North American type of construction most vulnerable to earthquakes. Paradoxically, many of those buildings house key infrastructure or governmental activities, and others would need to be operational in postdisaster situations.

The problem is particularly severe in eastern Canada where the generally lower level of seismic activity instills a lack of earthquake awareness. Consequently, recommendations to review the seismic resistance of existing structures meets considerable resistance, even for key postdisaster critical structures. Owners typically fear that engineers would perform seismic-resistance adequacy evaluations based on conservative analytical assumptions, mostly disregarding the potentially favorable contribution of structural or nonstructural elements whose effects cannot be quantified. For example, century-old buildings can sometimes be found to have no or little "theoretical" resistance to wind when analyzed according to accepted modern structural engineering practice and procedures, whereas the loading history of such structures actually demonstrates otherwise.

Although the Loma Prieta earthquake has again demonstrated the vulnerability of older structures to even moderate seismic excitations, concerned owners must also reconcile the potential need for expensive structural retrofitting with the levels of seismic risk and consequences of inaction. Clearly, for the mitigation of the seismic hazards to proceed effectively in Canadian regions of low to moderate seismicity, the adequacy evaluation of existing structures must be closely interrelated to the reliability and performance level expected of the

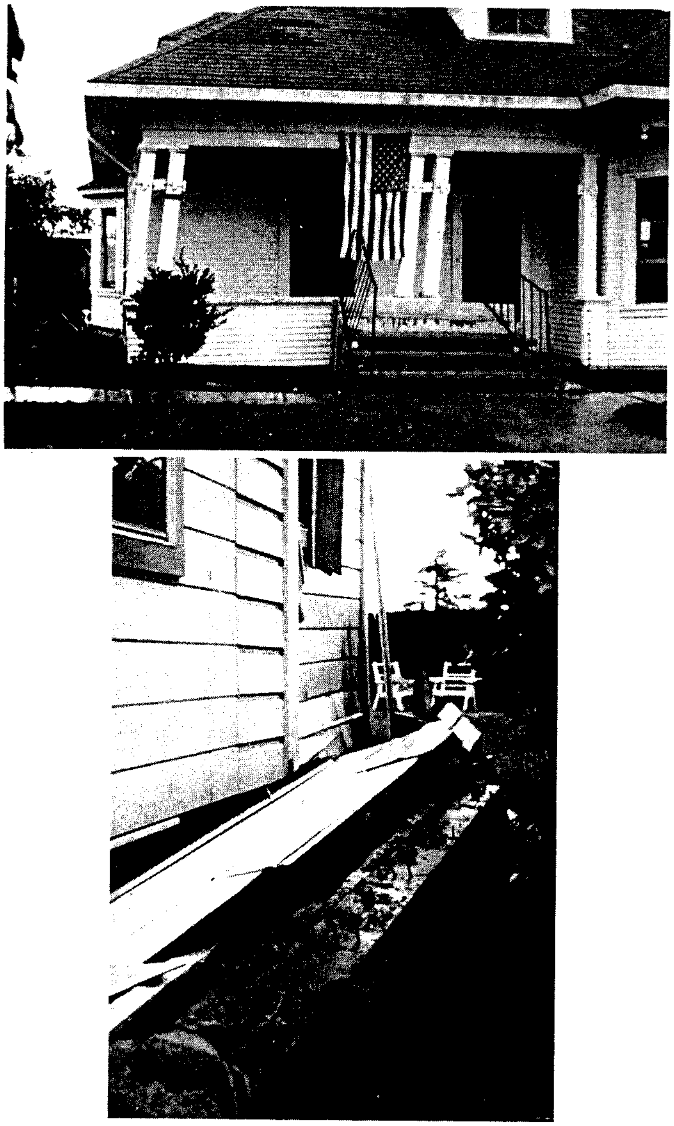

FIG. 17. Older residential structure on "pony-walls," not properly connected to its foundation (Hollister).

targeted facililties by owners, as well as the various probabilities of earthquake occurrence. In that perspective, the prime engineering goal would be to synthesize a realistic, neither unduly conservative nor permissive, statement of the seismic resistance capacity and ductility of the structural as well as architectural components of the buildings. Conducting all steps of the seismic adequacy evaluation using conservative engineering analysis and design assumptions will make this goal unattainable. Similarly, overestimating the actual capacity of each structural or nonstructural element to resist damaging cycles of seismic excitation could lead to a false and dangerous sense of security. The challenge lies in establishing a realistic analytical model of the system and providing the most accurate and reliable assessment of its actual capacity balancing the views obtained from the buildings' recorded performance history and the results from state-of-the-art analysis techniques.

The tools to allow owners to unambiguously specify their earthquake-survivability requirements at various reliability levels, and assisted by the technical recommendations of engineers, must be developed through further research. In parallel, a methodology for the seismic adequacy review of Canadian existing structures, based on the aforementioned premises, should also be developed and presented in a format that can lead to its adoption and implementation by regulating bodies. Guidelines to assist owners of large inventory of structures in priorizing their conduct of large-scale seismic adequacy reviews should be integrated to such a methodology. In addition to structural engineering considerations, geotechnical factors must be included in such a priority list. Micro-zonation maps, delineating zones where soils are expected to have various levels of detrimental influence during earthquakes, such as those developed for Quebec City (Chagnon and Doré 1987), would greatly assist in that respect.

The data collected from this earthquake should also be considered of uppermost importance for any serious Canadian heritage or historical structures preservation efforts. Typically, the special historical, architectural, and construction characteristics of heritage buildings restrict the extent of the permissible seismic retrofitting interventions, and a refined evaluation of the seismic capacity of such structures, along the lines previously enunciated, can be most advantageous. Toward that goal, and as indicated previously, comparative studies of typical Canadian heritage structures and those damaged by the Loma Prieta earthquake would enable the establishment of vulnerability thresholds for typical types of heritage of historical structures. In addition, special legislation mandating that seismic safety considerations be included as part of other preservation efforts would also be desirable.

The "soft-story" undesirable behavior exhibited by buildings of the Marina district is not a unique California feature. As previously mentioned, numerous buildings have been constructed with similarly soft first story throughout Canada, in engineered as well as nonengineered buildings, and will continue to be constructed as building codes currently do not address this problem. The earthquake-resistant behavior of the types of soft-story buildings most popular across Canada should be assessed, considering their various local seismic exposure.

Finally, it is unfortunate that earthquake awareness is generally deficient throughout the country. Private building owners' general disinterest about the seismic safety of their facilities only reflect the population's lack of awareness. Public decision makers, while more sensitive to those issues, are mostly under severe budgetary restrictions. Clearly, an effective policy of mitigation of seismic hazards will first require a major increase of the population's level of earthquake awareness. The next major Canadian earthquake to occur near a major urban center will definitely be of uppermost interest.

5. Conclusions

The recent October 17, 1989 Loma Prieta earthquake will obviously be closely studied by all parties involved in improving the general level of earthquake preparedness in North America. Being only of Richter magnitude 7.1 and producing significant structural damage as far as 80 km from the epicenter where base-rock peak ground accelerations would have ranged from $0.10g$ to $0.15g$, this earthquake provides a taste of what is to be at least expected in many Canadian seismic regions; the study of structural damage produced by this earthquake should be most pertinent to Canadian structural engineering practice.

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