Performance of masonry structures during the 1994 Northridge (Los Angeles) earthquake

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Abstract: The surface magnitude 6.8 Northridge earthquake which struck the Los Angeles area on January 17, 1994, damaged a large number of engineered buildings, of nearly all construction types. As earthquakes of at least similar strength are expected to occur in most of eastern and western Canada, the study of the effects of this earthquake is of particular significance to Canada. This paper, as part of a concerted multi-paper reporting effort, concentrates on the damage suffered by masonry buildings during this earthquake, and explains why the various types of observed failures occurred. The seismic performance of all masonry construction similar to that commonly found in Canada is reviewed, but a particular emphasis is placed on providing an overview of damage to unreinforced masonry structures which had been rehabilitated before this earthquake. To provide a better appreciation of the impact of this earthquake on masonry buildings, and a better assessment of the engineering significance of their damage in a Canadian perspective, this paper first reviews the evolution of building code requirements for unreinforced masonry buildings up to the seismic retrofit ordinances enacted prior to this earthquake. Examples of various damage types, as observed by the author during his reconnaissance visit to the stricken area, are then presented, along with technically substantiated descriptions of the causes for this damage, and cross-references to relevant clauses from Canadian standards and codes, as well as the recently published Canadian Guidelines for the Seismic Evaluation of Existing Buildings, whenever appropriate.

Key words: earthquake, unreinforced masonry, seismic rehabilitation, retrofit, retrofitted masonry building, reinforced masonry, buildings, failure, collapse, heritage buildings.

Résumé : Le tremblement de terre de Northridge, qui a eu lieu le 17 janvier 1994 dans la région de Los Angeles et dont l'amplitude sur l'échelle Richter était de 6,8, a endommagé un grand nombre de bâtiments de tous genres. Puisque des tremblements de terre d'une intensité semblable sont susceptibles de se produire dans la plupart des régions de l'est et de l'ouest du Canada, l'étude des effets de ce séisme est d'une importance indéniable pour le Canada. Dans le cadre d'une série d'articles sur le tremblement de terre de Northridge, cette communication porte une attention particulière aux dommages subis par les bâtiments de maçonnerie durant le séisme et explique pourquoi les divers types de défaillance se sont produits. La performance sismique de toutes les constructions de maçonnerie semblables à celles que l'on trouve un peu partout au Canada est examinée; cependant, l'auteur s'efforce de tracer un portrait des dommages subis par les constructions de maçonnerie non armée qui avaient fait l'objet d'une réhabilitation avant le séisme. Afin de mieux apprécier l'impact de ce tremblement de terre sur les bâtiments de maçonnerie et de mieux évaluer l'importance de leurs dommages d'un point de vue canadien, cet article examine d'abord l'évolution des exigences des codes du bâtiment pour les structures de maçonnerie non armée et ce, jusqu'aux ordonnances d'amélioration parasismique décrétées avant ce tremblement de terre. Des exemples de types de dommage observés par l'auteur à l'occasion d'une visite de la zone sinistrée sont ensuite présentés, ainsi que des descriptions techniques des causes de ces dommages. Enfin, des renvois aux alinéas pertinents des codes et normes du Canada et, le cas échéant, aux Lignes directrices canadiennes pour l'évaluation sismique des bâtiments existants nouvellement publiées, sont effectués.

Mots clés : tremblement de terre, maçonnerie non armée, réhabilitation parasismique, amélioration, bâtiment de maçonnerie amélioré, maçonnerie armée, bâtiments, défaillance, effondrement, bâtiments d'intérêt patrimonial.
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Introduction

On January 17, 1994, a severe earthquake measuring 6.4, 6.7, and 6.8 respectively on the Richter, moment, and surface magnitude scales (upgraded by the National Earthquake Information Center from the originally reported surface magnitude value of 6.6) hit the Los Angeles area, its epicentre being located in the San Fernando Valley community of Northridge. A number of engineered buildings, of nearly all construction types, suffered structural damage from this earthquake (EERI 1994*a*; CAEE 1994; EERC 1994; EQE 1994; and other papers in this special issue of the Canadian Journal of Civil Engineering). Still, relatively few modern engineered buildings suffered catastrophic and unrepairable damage when compared against the total building inventory of metropolitan Los Angeles; this is true even in the near epicentral region. Although some engineers may find this surprising, particularly given the severity of recorded ground accelerations in the epicentral area (Shakal et al. 1994; Porcella et al. 1994), a different outcome would have actually raised serious concerns either on the quality of the North American construction industry or on the state of earthquake-resistant design knowledge as expressed in recent building codes used throughout California. The excellent level of earthquake awareness in California is also largely accountable for this satisfactory performance. Indeed, the Los Angeles area was (and is still) bracing itself for either a Richter magnitude 8 along the San Andreas fault located approximately 20 km from the San Fernando valley, or a larger than magnitude 7 along the Elysian Park hidden trust fault system which crosses downtown Los Angeles. Both are capable of generating seismic excitations of much longer duration and higher intensity than the January 17 seismic event. Thus, in anticipation of these predicted major earthquakes, many owners had already shown wisdom and initiative and had their buildings seismically rehabilitated prior to the Northridge earthquake.

Nonetheless, the extent of structural damage produced by this earthquake is not insignificant, and a number of important lessons in earthquake-resistant design can be learned from damage observations and from investigations of the seismic performance of selected buildings. Indeed, numerous deficiencies in the current design and construction practice have been exposed by this earthquake, both for new and recently retrofitted structures of every building material.

Damage was particularly extensive in older buildings designed in absence of (or to inadequate) seismic-resistant design requirements, such as unreinforced masonry buildings which had not yet undergone seismic strengthening at the time of the Northridge earthquake. Existing unreinforced masonry buildings are undoubtedly most vulnerable to earthquakes, and constitute a serious seismic risk in the current North American urban environment. The Northridge earthquake provided an excellent opportunity to observe their seismic performance in contrast to similar rehabilitated structures over an extensive inventory of such existing buildings.

The main objective of this paper is to illustrate the damage suffered by masonry buildings during this earthquake, and to explain why the various types of observed failures occurred. Emphasis is on the performance of nonrehabilitated and rehabilitated unreinforced masonry buildings. To provide a better appreciation of the impact of this earthquake on masonry buildings, and a better assessment of the engineering significance of their damage in a Canadian perspective, this paper first reviews the evolution of building code requirements for unreinforced masonry buildings up to the seismic retrofit ordinances enacted prior to this earthquake. Examples of various damage types, as observed by the author during his reconnaissance visit to the stricken area (a visit started roughly 30 hours after the main shock), are then presented, along with technically substantiated descriptions of the causes for this damage. It is noteworthy that the author also later visited some of the Building and Safety Division Offices in cities where masonry buildings suffered damage, to consult numerous structural drawings and design calculations filed there. Some of the information gathered there has been instrumental to this paper. Since the generally poor seismic performance of unreinforced masonry buildings is already well known, as are the causes of this damage (Bruneau 1994*a*), only a few instances of damage to such buildings will be reported herein. Instead, emphasis here will be largely on providing an overview of damage to unreinforced masonry structures which had been rehabilitated before this earthquake. Finally, the performance of reinforced masonry structures in the epicentral region will also be briefly reviewed. In all cases, relevant clauses from Canadian standards and codes, as well as the recently published Canadian Guidelines for the Seismic Evaluation of Existing Buildings (CGSEEB) (NRC 1992), are referenced whenever appropriate for the convenience of the reader.

Seismological and geotechnical considerations, which are addressed thoroughly elsewhere, and the seismic performance of adobe-type masonry constructions, are beyond the scope of this paper. Techniques to stabilize and repair seismically damaged unreinforced masonry buildings will be reviewed in a future paper.

Historical overview

General

To understand the patterns of damage to masonry structures due to the January 17, 1994, Northridge earthquake, it is important to review the history of masonry design practice in the Los Angeles area. At the turn of the century, this practice was essentially identical to that followed uniformly throughout North America. However, in California, the design practice changed dramatically following the March 10, 1933, Long Beach earthquake of Richter magnitude 6.3 (Iacopi 1981; Yanev 1991; Moore 1986). This earthquake produced damage in Long Beach and surrounding communities in excess of $42 millions in 1933 dollars (more than $400 millions in 1995 dollars), and the death toll exceeded 120 (Alesch and Petak 1986; Iacopi 1981). Postearthquake investigation teams at that time reported that more than half of the 3417 damaged buildings in Long Beach were of unreinforced masonry construction (Alesch and Petak 1986). It was significant that a large number of these unreinforced masonry buildings which suffered damages were schools, and the total number of casualties and injuries would undoubtedly have been considerably larger had this earthquake not occurred at 5:54 p.m., a time when schools were fortunately empty. Although the seismic vulnerability of unreinforced masonry had long been recognized in California prior to 1933, with numerous reported examples of catastrophic damage during prior earthquakes (Iacopi 1981; Hansen and Condon 1989; Yanev 1991), the Long Beach earthquake abruptly showed to the general public how nearly all the children of a given locality could be suddenly killed or maimed by failing unreinforced masonry buildings. This provided the necessary political incentive to develop at once the first seismic-resistant design regulations, and simultaneously prohibit the construction of unreinforced masonry buildings. Before the end of 1933, most cities in California forever banned unreinforced masonry constructions; this was done on October 6, 1933, in Los Angeles.

The ban on unreinforced masonry buildings rapidly spread throughout the seismic regions of the western United States and, eventually, Canada. The 1975 edition of the National Building Code of Canada (NBCC) was the first to explicitly prohibit the construction of unreinforced masonry buildings in moderate to severe seismic regions (NRC 1975), and this is still enforced by clause 4.1.9.3.(6) of the 1990 edition of the NBCC (NRC 1990) in seismic velocity zone 2 or greater, i.e., where more than 50% of the Canadian population lives.

Los Angeles unreinforced masonry rehabilitation ordinances

While the aforementioned ban was effective in preventing further construction of these seismically hazardous buildings, California was left with a considerable inventory of unreinforced masonry buildings, with approximately 25 000 such buildings still in existence at the beginning of the 1990s (Seismic Safety Commission 1991*b*). Thus, not surprisingly, many unreinforced masonry buildings were severely damaged during every subsequent moderate earthquake to hit that state (Murphy 1973; Iacopi 1981; Reitherman et al. 1984; Shah et al. 1984; Swan et al. 1985; Hart et al. 1988; Yanev 1991). These numerous failures acted as constant reminders of the threat posed to life by these seismically hazardous buildings.

Since nearly half of the existing Californian unreinforced masonry buildings are located in southern California, largely in the Los Angeles metropolitan area, the political will to mitigate this seismic hazard appeared there as early as the 1940s, first in Long Beach and Los Angeles. For example, Los Angeles adopted a "parapet correction ordinance" in 1949 requiring that owners remove, brace, or strengthen the parapets which could fail and fall on pedestrians during an earthquake. However, although a much more comprehensive seismic rehabilitation program was needed to effectively mitigate the seismic risks from unreinforced masonry buildings, numerous political, economical, social, and technical factors dampened this desire to act. A comprehensive historical description of the challenges met by proponents of seismic risk mitigation policies, and of the compromises which were needed to enact enforceable policies, has been well documented by other researchers (Alesch and Petak 1986).

Eventually, following numerous public meetings, compromises, and delays from lawsuits, an enforceable ordinance, now known as Division 88 of the Los Angeles Building Code, was adopted by the city of Los Angeles and became effective February 13, 1981 (Moore 1986). It is noteworthy that the very first paragraphs of this ordinance clearly state: "The purpose of this division is to promote public safety and welfare by reducing the risk of death or injury that may result from the effects of earthquakes on unreinforced masonry bearing wall buildings constructed before 1934. Such buildings have been widely recognized for their sustaining of life hazardous damage as a result of partial or complete collapse during past moderate to strong earthquakes. The provisions of this division are minimum standards for structural seismic resistance established primarily to reduce the risk of life loss or injury *and will not necessarily prevent loss of life or injury or prevent earthquake damage to an existing building which complies with these standards*" (emphasis added by the author). Clearly, with such an explicit disclaimer, conscientious and responsible structural engineers have typically informed owners of the implicit level of protection purchased by the mandatory seismic rehabilitation work, in order to unambiguously limit their professional liability, and also because nothing in the rehabilitation process, once triggered, prevents individual owners from seeking a higher level of seismic protection. It has been reported that the differential cost to purchase this additional safety and (or) superior performance is sometimes negligible compared with the initial investment (Moore 1986).

It is noteworthy that the City of Los Angeles also required that unreinforced masonry buildings that served an essential function had to be evaluated and upgraded for full compliance to the seismic-resistant requirements for new buildings, a considerably more stringent requirement than that of Division 88. Incidentally, as the City of Los Angeles was the primary owner of unreinforced masonry essential buildings, this also demonstrated the conviction and sincerity of public officials toward the reduction of seismic risks.

There existed 8242 potentially hazardous unreinforced masonry buildings in the City of Los Angeles when it passed its 1981 seismic retrofit ordinance. At the time of the Northridge earthquake, 1617 of those had been demolished, partly for seismic-related reasons, but also because of the normal attrition process also found in most other North American cities. Of the remaining lot, more than 6000 had been rehabilitated to be in full compliance with Division 88 by January 1994. Most of the still nonrehabilitated unreinforced masonry buildings are located in south central Los Angeles, a part of Los Angeles relatively far from the epicentre of the Northridge earthquake and, thus, where no recorded damage occurred.

Clearly, Division 88 of the Los Angeles Building Code was a pioneering ordinance. It provided a detailed prescriptive seismic evaluation and rehabilitation procedure, further complemented by technical guidelines periodically published by the City's Department of Building and Safety, Earthquake Safety Division. Moreover, subsequent editions of Division 88 incorporated numerous additions, changes, and enhancements. A detailed review of the evolution of Division 88 is beyond the scope of this paper.

It is noteworthy that, starting in 1988, a special procedure (known in Los Angeles as the Rule of General Application) based on research on the behaviour of unreinforced masonry buildings (ABK 1984) was provided as an alternate to the general procedure used previously. A comprehensive review of this special procedure is available elsewhere (Bruneau 1994*b*; Structural Engineers Association of Southern California 1986, 1991). For the present paper, it is suffice to know that, to evaluate the seismic adequacy of existing unreinforced masonry buildings, both procedures promote the use of an equivalent static lateral force level lower than that used for new buildings, provided that structural integrity is ensured and that measures are taken to mitigate the risk of failures known to be detrimental to life safety. Thus, both procedures require tests to control the quality of the existing masonry and obtain material properties; anchorage of walls to floors and roof; verification of the dynamic stability of walls and parapets, and reinforcement if needed to prevent their out-of-plane failure; and verification and reinforcement if needed of the in-plane resistance of walls and piers. However, the special procedure also controls the nonlinear dynamic characteristics of wood floor diaphragms which excite walls in their out-of-plane direction, and allows consideration of the in-plane rocking-resistance. In Los Angeles, although some structural engineers are more comfortable with the general procedure, approximately 50% of the unreinforced masonry seismic rehabilitation projects since 1988 have used the special procedure.

The Los Angeles ordinance eventually provided the model for Appendix Chapter 1 of the Uniform Code for Building Conservation (UCBC) (ICBO 1991*a*), itself adopted by the California Building Seismic Safety Commission (BSSC) as a model code for the seismic evaluation and rehabilitation of existing buildings (Seismic Safety Commission 1991*a*), the ATC-22 (ATC 1989), a similar NEHRP document (FEMA 1992), and finally, with minor modifications to be compatible with Canadian design practice, Appendix A of the recently published Guidelines for the Seismic Evaluation of Existing Buildings (NRC 1992).

Santa Monica and Culver City

It is worthwhile to review in detail the state of seismic retrofit activities, at the time of the Northridge earthquake, in two other cities where severe damage to masonry buildings occurred: Santa Monica and Culver City.

The City of Santa Monica first initiated a semivoluntary seismic rehabilitation program in 1978. Typically, the City would issue "Notice of Substandard and Potentially Hazardous Buildings" to owners of unreinforced masonry buildings. These documents were kept on file at City Hall, along with legal descriptions of the buildings and the name of the owners. They were intended to trigger semivoluntary seismic rehabilitation work when ownership of buildings would change hands, as financial institutions would likely hesitate to issue mortgages on buildings with such clearly identified liabilities. A certificate of termination of the above notice would obviously be issued upon seismic rehabilitation of a building. However, to accelerate the seismic risk mitigation process, on July 25, 1990, Santa Monica enacted an ordinance requiring that owners of unreinforced masonry buildings have structural engineers assess the seismic resistance of their buildings. For each building, a form provided by the City had to be filled by the retained structural engineer. It included a "statement of minimum estimated level of seismic resistance of existing unreinforced masonry to failure/collapse," in which the engineer on record attested "I certify that I have personally inspected the subject building and performed a comprehensive evaluation pursuant to the standards of Chapter 23 of the Uniform Building Code. In my professional opinion the MINIMUM lateral load resisting capacity of the existing structure as a percentage (%) of gravity acceleration is estimated to be ____%." For most buildings in Santa Monica, this capacity was estimated to be between 0 and 3%, although a few reported values were as high as 7%. Calculations were to be provided and filed at City Hall.

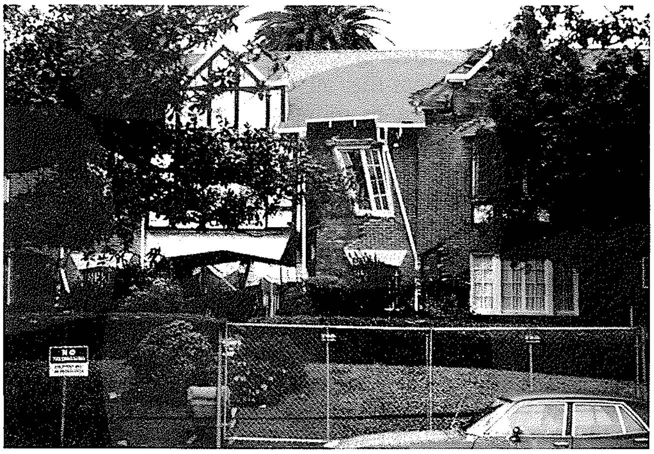
Finally, on September 29, 1992, in answer to the California State requirement that all municipalities adopt a proposed model ordinance to mitigate the seismic risk from potentially hazardous buildings, the City of Santa Monica adopted Appendix Chapter 1 of the 1991 UCBC as their Seismic Retrofitting Code. A dual timetable for compliance was proposed: rehabilitation work was to be complete in less than 2 to 5 years, depending on the risk classification of the building (risk being established as a function of occupancy), or 4 to 10 years if wall anchors were installed and hazardous parapets were braced, strengthened, or removed within a year. Consequently, Santa Monica's comprehensive and mandatory program for the seismic-rehabilitation of unreinforced masonry buildings was still underway at the time of the earthquake, with 82 buildings still nonretrofitted, 12 buildings partially retrofitted (wall anchors and parapet bracing only), and 128 buildings retrofitted to be in full compliance with Appendix Chapter 1 of the UCBC.

Culver City, a small city almost completely surrounded by Los Angeles on all sides, also adopted a seismic risk mitigation program essentially identical to the Los Angeles Division 88, as early as February 9, 1987. Thus, 59 of the 65 existing unreinforced masonry buildings in Culver City had fortunately already been retrofitted before the Northridge earthquake. It is noteworthy that many residential house unreinforced masonry chimneys in Culver City failed at the roof line during the 1971 San Fernando earthquake, and were repaired by adding a new reinforced masonry section on top of the remaining unreinforced masonry base. As expected, these unreinforced masonry bases failed during the Northridge earthquake. This provided the City with an opportunity to include, in its Post-Disaster Recovery and Reconstruction Ordinance of April 4, 1994, a requirement that all damaged unreinforced masonry chimneys be demolished to their foundation. To the author's knowledge, it is the only such existing chimney ordinance.

Nonstructural masonry

All the aforementioned seismic risk mitigation ordinances solely address those risks posed by unreinforced masonry buildings, i.e., buildings having unreinforced masonry bearing walls. Although the failure of nonstructural masonry will not jeopardize the structural integrity of a building, the killing potential of falling masonry is obviously not a function of its structural value prior to failure. To this day, the severe seismic hazard created by failing nonstructural components, such as masonry veneers and cladding in general, has not received the attention it deserves from the structural engineering community, even though these can be as life-threatening as structural failures. It is often alleged by some engineers that seismic-resistant design of nonstructural components is beyond their scope of work (unless specified otherwise contractually and remunerated accordingly), and is rather a responsibility of the architect. Indeed, the responsibility for the seismic-resistant design of nonstructural components is, at best, ill defined (Cohen 1991; Bruneau and Cohen 1994).

Fig. 1. In-plane damage to nonstructural masonry veneer, Ye Old San Vincente Apartments, Santa Monica.



Not surprisingly, failures were numerous during the Northridge earthquake.

Many wood-frame residential constructions having architectural masonry finishes and veneers suffered considerable damage during this Northridge earthquake. Generally, poor connection and flexibility mismatch between the wood framework and masonry typically resulted in severe damage to the nonstructural masonry. In cases of continuous veneers, as in some types of residential construction, the veneer itself can attract a considerable portion of the seismic force, depending on the relative rigidities of the masonry and wood walls. Even when the veneer is poorly connected to the structure, it may be engaged into a joint seismic-response mode by setbacks, irregular floor plans, and other geometric features. Such veneers may subsequently experience severe in-plane failure, as shown in Fig. 1.

Equally dramatic in-plane X-cracking is visible on the spire of the First Catholic Church of Santa Monica (Fig. 2). Although accidentally initially identified as an unreinforced masonry building by the City of Santa Monica when issuing notices of substandard and potentially hazardous buildings, a structural engineering investigation later revealed this church to be a reinforced concrete structure clad with special Arizona limestone blocks "interlocked" together. Severe cracking of the masonry veneer also occurred elsewhere throughout the church, and out-of-plane failure of one loadbearing unreinforced masonry wall also occurred. Even though the visible damage suggests an unreinforced masonry tower in a state of eminent collapse, the structural integrity of this tower has actually not been affected by this earthquake. In fact, somewhat similar but milder damage was suffered by the tower during the 1987 Richter magnitude 5.9 Whittier earthquake epicentered approximately 35 km east of Santa Monica. This severely cracked masonry veneer had been restored then, and was undergoing repairs anew at the time of this writing. However, without an engineered solution properly considering the relative rigidities of the veneer and structural system, similar damage in future earthquakes is likely to recur.

Many slender anchored veneers improperly connected to their backup structure have failed in an out-of-plane direction (e.g., Fig. 3; see also TMS 1994). The 1991 edition of the Uniform Building Code (UBC) specifies masonry veneer anchorage requirements for severe seismic zones (ICBO 1991*b*). Unfortunately, these requirements are in Chapter 30 of the UBC, i.e., a chapter outside Part V of the UBC where all engineering regulations are gathered. For the stone veneer of the building shown in Fig. 3, over wood studs spaced at 400 mm (16 in.), code-compliant anchorage would have been provided by horizontally placed No. 9 gauge (imperial size) reinforcement wires engaged into lips on the extended legs of gauge 14 corrugated sheet metal anchors spaced at 400 mm (16 in.) horizontally and 300 mm (12 in.) vertically. Most existing buildings do not have such anchorage. Clause 4.1.9.3.(6) of the NBCC requires that masonry nonloadbearing walls and partitions be reinforced in velocity- or acceleration-related zones 2 and higher, but exempts walls of 200 kg/m^2 or less which are less than 3 m tall. It is noteworthy that a one-wythe veneer of normal density brick weighs just slightly less than 200 kg/m^2. Reinforcement requirements for nonloadbearing walls are specified in Clause 5.8.2 of the CAN3-S304-M84 Canadian standard. Veneers should be tied to their backing in accordance with Clause 9.20.9.5 of the 1990 edition of the NBCC, and the CAN3-A370-M84 standard (CSA 1984).

Adhered veneers are very popular in California. These veneers consist of thin-masonry bonded to their backup structural material. They generally performed satisfactorily during this earthquake (TMS 1994).

Failures of some hollow clay-tile nonstructural infills or partitions, not reinforced and not properly tied to their structure, also occurred during the Northridge earthquake. Typically, during earthquakes, such infills separate from the swaying buildings, and are vulnerable to out-of-plane failure. Examples of this unsatisfactory behaviour are shown in Figs. 4 and 5.

Finally, unbraced heavy masonry masses protruding above roof level (or ground level) are known to be particularly vulnerable to earthquakes. Hence, a phenomenal number of masonry chimneys and property line walls collapsed during the Northridge earthquake, as would be typically expected during any earthquake of this severity. To the author's knowledge, no specific requirement exists mandating the bracing of existing unreinforced masonry chimneys or specifying how this must be done. However, common sense dictates that their vulnerability should not be overlooked, and parapet bracing requirements could be extended to chimneys during a seismic rehabilitation.

Unreinforced masonry buildings

Since massive urban development in the San Fernando Valley fortuitously occurred mostly after 1933 (Pegrum 1964), only a few unreinforced masonry buildings, sometimes of archaic adobe-type construction, had been built in the San Fernando Valley before measures were in place proscribing their construction there. Moreover, except for those located at the southern end of the valley, nearly all of these few unreinforced masonry buildings located in the valley were destroyed or severely damaged during the 1971 Richter magnitude 6.4 earthquake which had its epicentre only 25 km (15 miles) north of the Northridge one. These failures have been well documented elsewhere (Murphy 1973). Among those failures,

Fig. 2. Damage to nonstructural Arizona limestone cladding of First Catholic Church of Santa Monica: (*a*) overall view of facade; (*b*) closeup view of X-type shear-cracking cladding damage of tower.

collapse of the Veteran's Hospital was a particularly dramatic example of the seismic life-safety hazards produced by unreinforced masonry buildings.

Thus, as far as unreinforced masonry buildings are concerned, the Northridge earthquake was felt much farther from the epicentre, mostly north and south, in the downtown core of older cities such as Santa Monica, Hollywood, Los Angeles, Culver City, Fillmore, and others, up to 30 km from the epicentre, where numerous older unreinforced masonry buildings exist. Ironically, many of the unreinforced masonry buildings damaged by this earthquake were scheduled for seismic rehabilitation within the year.

Rough estimates of effective peak accelerations, obtained by clipping the single or few isolated peaks of extreme peak ground acceleration often present in strong motion records, indicate that horizontal effective peak ground accelerations of $0.20g$, $0.25g$, and $0.30g$ occurred in Hollywood, North Hollywood, and Santa Monica. These correspond to approximately 70% of the peak ground values, except for Santa Monica where this corresponds to 30% of the recorded peak value. These are consistent with the observation that damage in unreinforced masonry buildings in California is typically triggered past the $0.15g$ to $0.20g$ effective peak ground acceleration for buildings having weak to good mortar respectively, when duration of strong shaking is less than 15 s (Schmid 1994).

The general modes of failure of unreinforced masonry buildings, repeatedly reported by earthquake reconnaissance teams in the past, usually belong to one of the following categories: (*i*) lack of anchorage; (*ii*) anchor failure; (*iii*) in-plane failure; (*iv*) out-of-plane failure; (*v*) combined in-plane and out-of-plane effects; and (*vi*) diaphragm-related failures. A detailed description of each has already been published (Bruneau 1994*a*) and will not be repeated here. However, the Northridge earthquake provided some additional striking examples worth reporting hereunder.

As is usually the case, out-of-plane failures were numerous. These pose a severe life-safety hazard, but fortunately, nobody died from falling masonry during the Northridge earthquake as it struck at 4:30 a.m., a time when people are away from the downtown cores where unreinforced masonry buildings are usually located. For example, a minor out-of-plane failure, such as the parapet failure on the alley side and back of Henshey's Men Shop in downtown Santa Monica, could have killed or maimed many as part of the debris and loose masonry fell across the store's main entrance (Fig. 6).

Parapets behave as cantilevers protruding from the roof level, cracking at that level and rocking over the cracked plane until a sufficiently strong jolt overturns them completely. Figure 7 illustrates this particular behaviour well in a building for which the east parapet has completely failed while the west parapet coincidentally only cracked and rocked in a stable manner without collapsing. The mechanisms of out-of-plane rocking and dynamic stability are discussed elsewhere (Bruneau 1994*b*).

A closer inspection of buildings that have suffered out-of-plane wall failures revealed that, in many instances, some anchors were present in the walls that failed. A frequently encountered type of archaic anchor, known in California as government anchors, is shown in Fig. 8. Clearly, these existing anchors provided insufficient restraint against the seismically induced forces, and masonry walls ruptured around the anchors. This is not surprising as government anchors, like many other types of old anchors, have never been designed

Fig. 3. Out-of-plane failure of nonstructural stone veneer of building near Woodman and Ventura avenues, Sherman Oaks, San Fernando Valley: (*a*) overall view of damage; (*b*) closeup view of failed veneer.

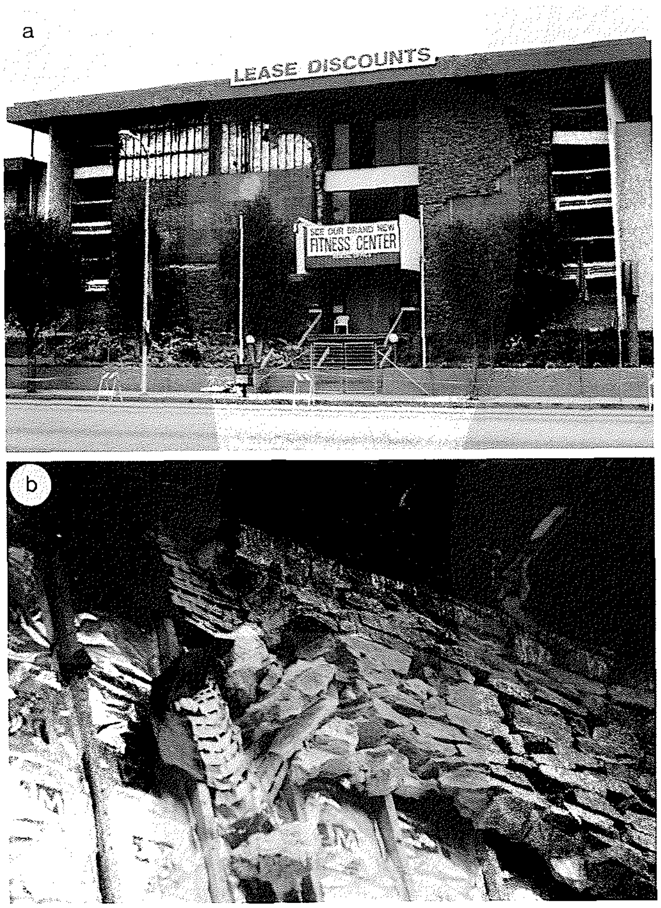
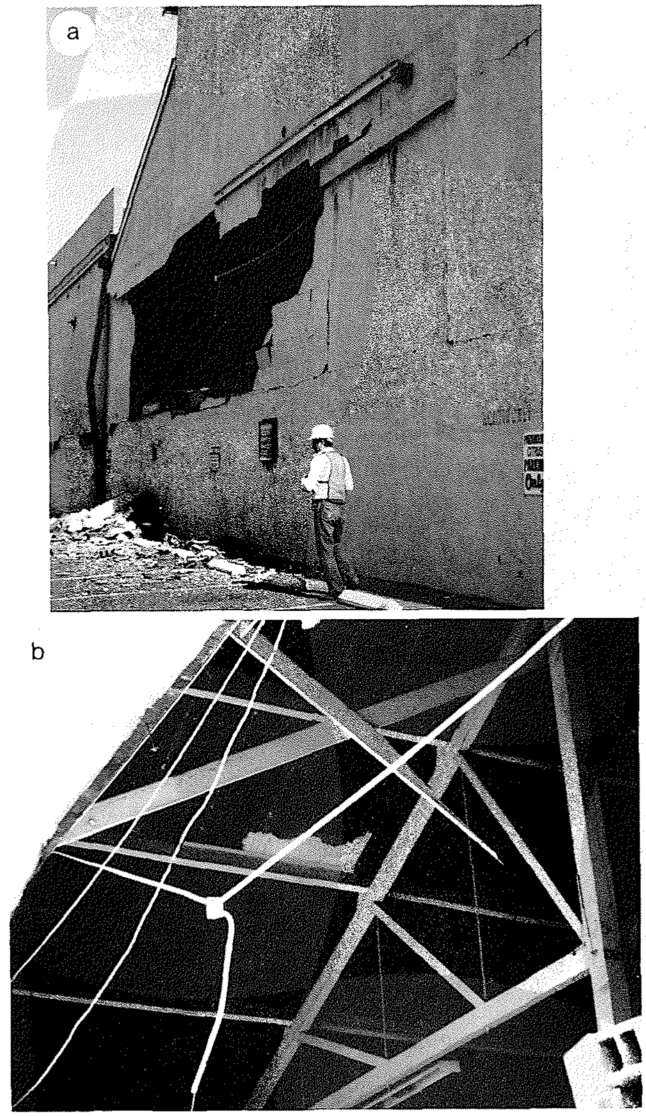


Fig. 4. Hollow clay-tile masonry infill failure, Paramount Citrus building, Mission Hills, San Fernando Valley: (*a*) overall view of infill separation and out-of-plane failure; (*b*) failure of timber bracing tying unreinforced masonry wall at the out-of-plane failure location.



or intended to provide earthquake resistance. Exterior wythes of multi-wythe walls also failed in an out-of-plane manner because of inadequacy (or sometimes absence) of the collar joint. In such a case, each wythe could behave as an individual slender one-wythe wall wherever the collar joint is deficient, and risks failure as shown in Fig. 9. It is noteworthy that header units, generally used in pre-1933 Californian unreinforced masonry constructions to provide connection between the wythes of multi-wythe walls, did not alone provide a sufficient mechanical connection between the wythes.

Although the majority of unreinforced masonry buildings damaged during this earthquake were in good condition, and of archaic but competent construction, a few obviously displayed extremely poor quality construction. Out-of-plane failure of such a shoddily constructed wall, where bricks and blocks have been used interchangeably without any systematic pattern and where both the head and collar joints are missing throughout, is shown in Fig. 10. In that example, the roof drainpipes embedded in the walls created additional weaknesses which helped precipitate failure.

Numerous in-plane failures of unreinforced masonry piers and walls also occurred. For example, the 400 Broadway building, in Santa Monica, suffered severe in-plane shear failure of many piers, including the corner piers (Fig. 11) which exhibited seismically induced X-type cracks in both orthogonal directions. Finally, some cases of apparent combined in-plane and out-of-plane failures were observed (e.g., Fig. 12).

A complete inventory of the extent of damage to unreinforced masonry buildings will take many months to compile, but it is noteworthy that for the first time on a large scale, comparisons between the seismic performance of nonrehabilitated and rehabilitated unreinforced masonry buildings excited by a moderate earthquake will be possible.

Rehabilitated unreinforced masonry buildings

Clearly, the majority of unreinforced masonry buildings rehabilitated (retrofitted) prior to the Northridge earthquake

Fig. 5. Failure of hollow clay-tile masonry infill in reinforced concrete frame, Appian Way, Santa Monica.

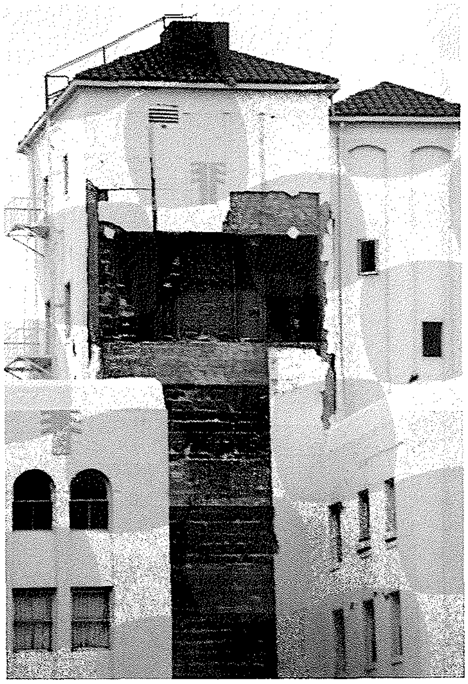
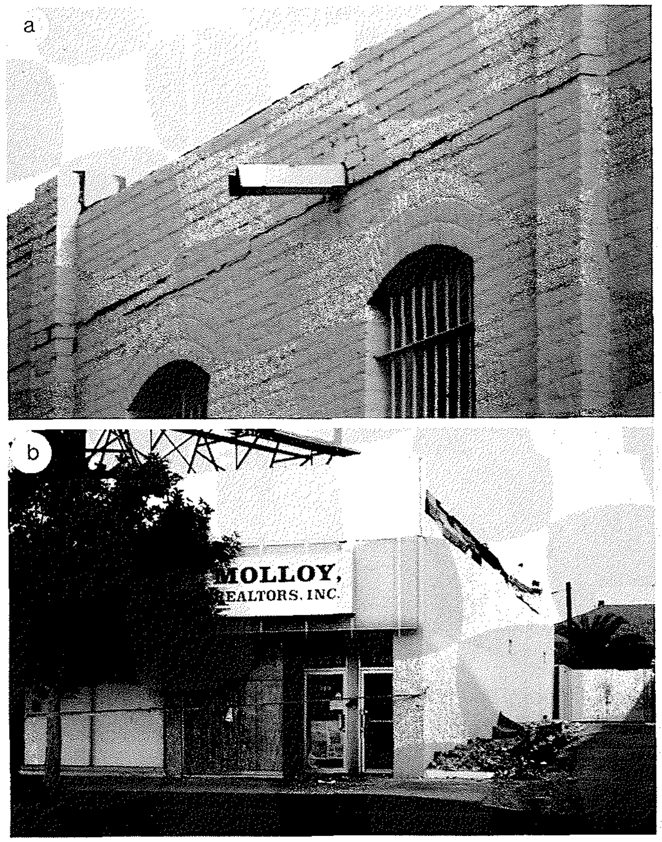


Fig. 6. Parapet failure, Henshey's Men Clothing Shop, Santa Monica Boulevard, Santa Monica.

survived undamaged. For example, in Fig. 13, which effectively illustrates this satisfactory performance, an undamaged retrofitted building is adjacent to (although separated by an alley-way on one side) two nonretrofitted unreinforced masonry buildings which suffered considerable damage during this earthquake.

However, many other buildings which had been thoroughly seismically retrofitted suffered some form of damage. Three months after the earthquake, the City of Los Angeles had identified 413 such damaged retrofitted buildings out of the more than 6000 structurally retrofitted to be in compliance with Division 88 of the Los Angeles Building Code, or with the special procedure provided by the Los Angeles Rule of General Application. This number is expected to reach

Fig. 7. Out-of-plane parapet failures in unreinforced masonry building, near Santa Monica Boulevard and Lincoln Avenue: (*a*) visual evidence of west wall parapet cracking and rocking at roof level; (*b*) collapsed east wall parapet.



450 buildings once all inspections are completed, with 200 having suffered moderate to severe damage. Statistics for other cities were not available.

Overall, the extent of damage to rehabilitated unreinforced masonry buildings is less than predicted by the consensus opinion of a panel of experts consulted prior to the Northridge earthquake (EERI 1994*b*). These experts estimated that damage to retrofitted unreinforced masonry buildings, located approximately 30 km (20 miles) from the epicentre of a magnitude 6 to 6.5 earthquake, would be such that approximately 4% of these buildings would completely collapse or be noneconomically repairable, 10% to 20% would suffer extensive structural and nonstructural damage requiring long-term building closures while awaiting extensive repairs, and 15% to 25% would suffer mostly nonstructural damage along with minor nonthreatening structural damage repairable within a few weeks or months.

The failures of retrofitted unreinforced masonry buildings during the Northridge earthquake can be attributed to a number of clearly identifiable causes, acting independently or concurrently. These are (*i*) poor quality masonry, including absence of good collar joints; (*ii*) nonanchoring of veneer wythe; (*iii*) improper consideration of veneer wythe as a structural wythe; (*iv*) nonrepresentative masonry strengths obtained from testing; (*v*) incomplete intermediate wall bracing system; (*vi*) flexible unanchored ceiling system pounding

Fig. 8. Collapsed unreinforced masonry north wall of First Christian Church of Santa Monica: (*a*) global view of out-of-plane failure; (*b*) closeup view of ineffective government anchor.

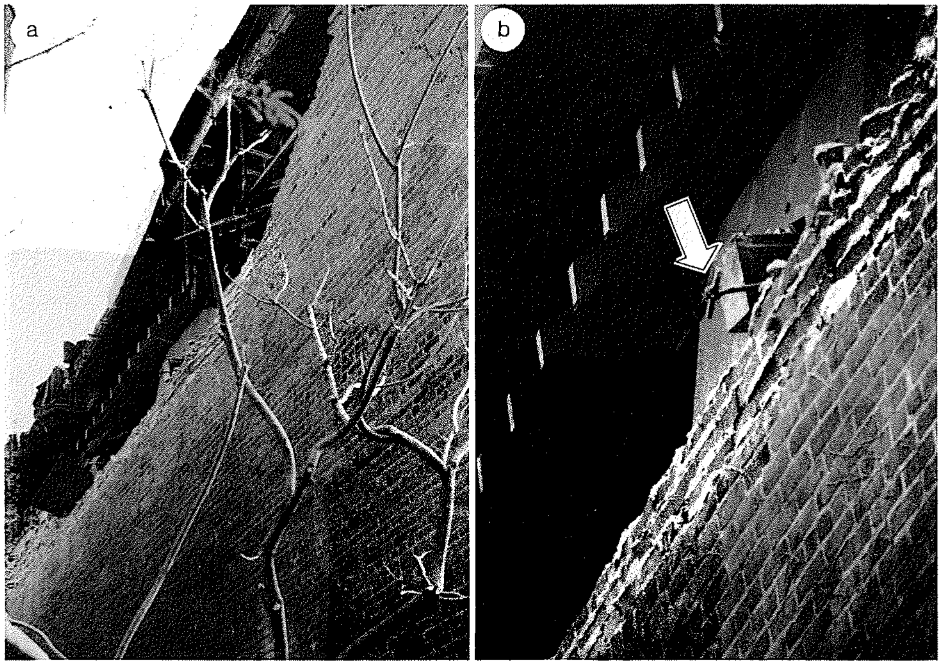


Fig. 9. Out-of-plane failure of exterior wythe of multi-wythe walls having poor-quality collar joint, near 4th Street and Santa Monica Boulevard, Santa Monica.

walls below roof line; (*vii*) incomplete or partial retrofits; and (*viii*) behaviours not explicitly addressed by existing seismic evaluation procedure requirements. These are reviewed in more detail hereafter, along with some examples of unsatisfactory performance during the Northridge earthquake. However, a comprehensive review of the seismic evaluation and rehabilitation philosophy and techniques generally followed by structural engineers in southern California is available elsewhere (Bruneau 1994*b*).

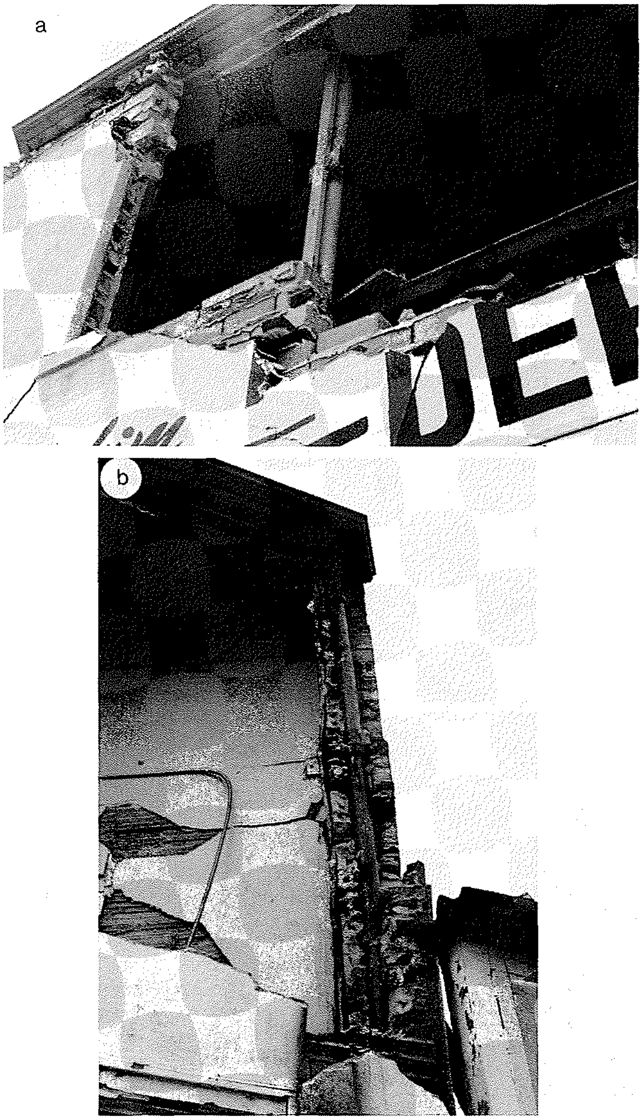
Poor quality masonry

A frequently observed failure in retrofitted building whose walls were properly anchored to floors and roof diaphragms was the out-of-plane failure of many courses of the outer wythe of masonry on one or several walls of a given building. In all cases, closer inspection of the wall revealed the absence of an adequate collar joint in the area where failure occurred. This is particularly a problem in three-wythe walls where localized areas of poor workmanship in the interior wythe could be hidden by well-executed outside wythes (Fig. 14). Therefore, during individual *in situ* push tests to obtain material properties, special attention must be paid to inspect whether an adequate collar joint is present. If mortar filling of this collar joint is absent or inadequate, each wythe risks being excited out-of-plane as an individual one-brick wide wall whose failure is significantly more likely to occur. If mortar quality is poor, some parts of outside wythes may also "slide" out of the wall. Clearly, it is possible that localized "pockets" of poor quality masonry may escape detection, in spite of thorough testing and inspection. However, if detected, all masonry that does not meet the specified standards should be removed and replaced (Clause A.5 of CGSEEB, or Section A106.(b) of UCBC). Note that the failure shown in Fig. 14 is minor whereas this unreinforced masonry building would have most certainly collapsed had it not been retrofitted prior to this earthquake.

Nonanchoring of veneer wythe

A common pre-1993 Californian unreinforced masonry construction practice was to construct a veneer wythe compositely with the bearing masonry wall by using mortared collar joints or by filling a small cavity space with grout. Mechanical connection of the exterior brick veneer to the interior wythes was nominal, when present at all. This is very different from the veneer construction commonly found in the eastern United States and Canada where a significant air space is left between the brick veneer wall and masonry

Fig. 10. Example of extreme poor quality unreinforced masonry wall construction, Lankershim Boulevard, North Hollywood: (*a*) irregular use of bricks and blocks, and missing head and collar joints; (*b*) embedded roof drainpipe weakening out-of-plane resistance.



backup wall, to act as a rain screen and to accommodate insulation, and where metal ties are used to connect the veneer to the backup wall.

Division 88 (and the UCBC) clearly requires that veneer anchors designed as per current requirements be installed if existing conditions are found to be deficient. Existing veneer anchor ties are judged acceptable only if they are corrugated galvanized iron strips no less than 25 mm (1 in.) wide by 200 mm (8 in.) long and 1.6 mm (1/16 in.) thick. They must be spaced at no more than 430 mm (17 in.) horizontally if present in every alternate course vertically, or no more than 230 mm (9 in.) if present every fourth course vertically (Section A110.(g) of the UCBC). However, it is unclear whether this requirement has been enforced. Veneer anchorage determination reports are not consistently filed at City Halls along with the engineer's retrofit calculations. Moreover, when these were filed and clearly demonstrated inadequate existing

Fig. 11. Severe in-plane shear failure of unreinforced masonry pier, Broadway and 4th Street, Santa Monica: (*a*) global view; (*b*) closeup view of seismically induced X-type shear cracking.

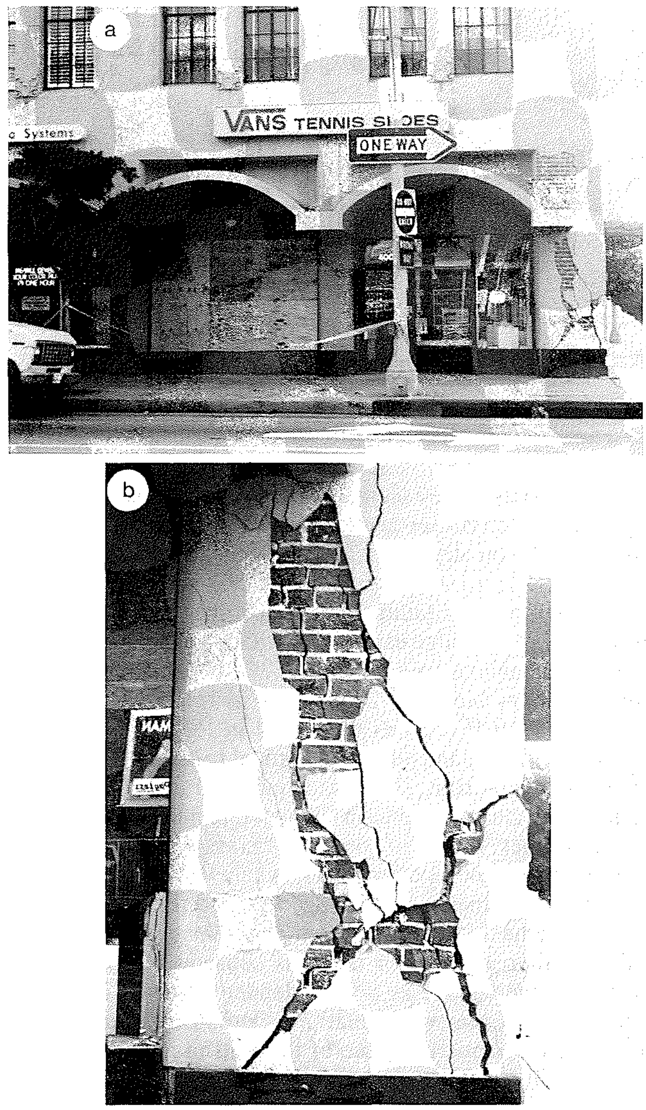


Fig. 12. Example of combined in-plane and out-of-plane failures, 3rd Street, Santa Monica.

Fig. 13. Undamaged retrofitted unreinforced masonry building located between two severely damaged unretrofitted ones, Santa Monica Boulevard, Santa Monica.

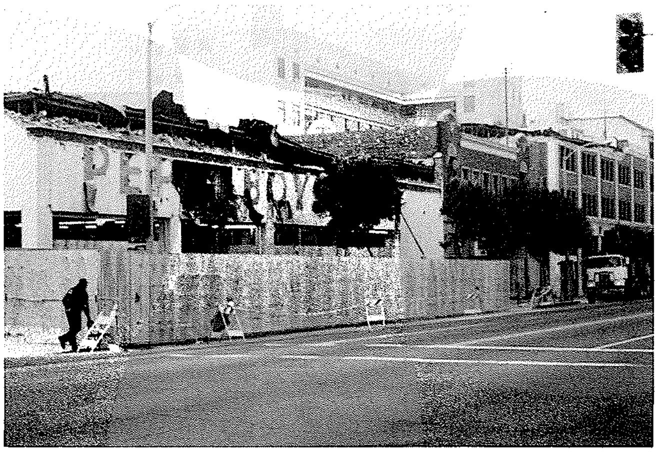


Fig. 14. Local out-of-plane failure of outer wythe of masonry walls of a retrofitted building on Washington Boulevard, Culver City: (*a*) view of wall, stabilized after the earthquake; (*b*) close-up revealing localized poor quality of workmanship in interior wythe of a three-wythe thick wall, particularly absence of collar joint where failure occurred.

veneer anchorage, they seem to have been ignored. Undoubtedly, the cost of veneer anchorage and the insignificance of veneer failure on structural survival may partly explain this situation.

Localized veneer failure on a retrofitted building which survived the Northridge earthquake without structural damage is shown, as an example, in Fig. 15. This one-story unreinforced masonry building, 21.6 m (71 ft) by 45.4 m (149 ft) in plan, built in 1923, was retrofitted in 1991 by anchoring walls at roof level, and adding a moment resisting steel frame along the open front. The inadequacy of existing veneer anchorage is clearly visible in Fig. 15*b*, and the out-of-plane failure of that veneer is not surprising. The absence of headers and of good collar joints, whenever encountered in an unreinforced masonry building, will effectively isolate the exterior wythe of masonry from the remaining structural wall. Whether intended or not, this consequently makes an exterior wythe behave as a very slender veneer, vulnerable to out-of-plane seismic excitation. Combined in-plane and out-of-plane seismic response of the wall may help precipitate this failure.

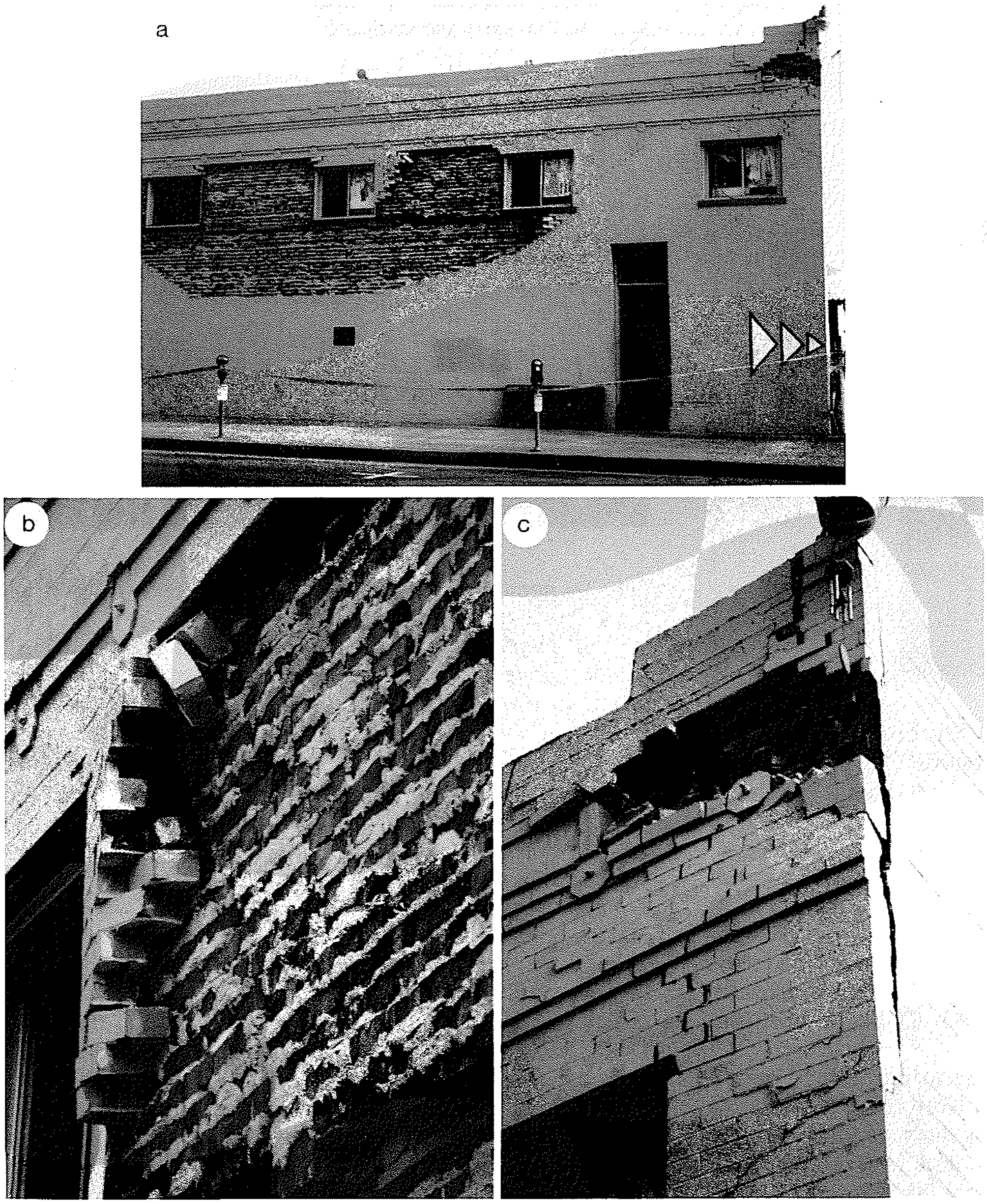
Improper consideration of veneer wythe as a structural wythe

Clearly, when seismic rehabilitation activities are performed, the dynamic stability of walls between anchored floors is checked against permissible limits of wall height-over-thickness ratio (h/t) developed based on results of dynamic tests (ABK 1984; ICBO 1991*a*; NRC 1992; Bruneau 1994*b*). These limits are derived for walls capable of behaving as a unit in their out-of-plane direction. As an extension to the previously mentioned problem, it is a mistake to include an exterior veneer wythe in the effective thickness used in the calculation of dynamic stability. Regular full-width headers must be present at specified intervals (Clause A5 of CGSEEB, Section A106.(c).2 of UCBC) in any given wythe before it may be considered as part of the effective thickness. Visual inspection is usually sufficient to determine the presence of a veneerlike wythe.

For example, if the damaged unreinforced masonry wall at the top story of the building shown in Fig. 16, which consists of a two-wythe brick wall and a veneer, is accidentally taken to be a three-wythe wall, its calculated h/t ratio becomes 12.7 instead of 18.4. While the latter result reveals the existence of an out-of-plane dynamic instability problem, and would lead to the addition of wall bracing, the former gives the illusion of dynamic stability when admissible cross-walls are present. This emphasizes the importance of a site visit by the structural engineer, particularly since wall thicknesses taken from existing drawings often only show the total wall thicknesses.

Incidentally, some structural engineers have alleged that two-wythe walls may not perform as intended, since the anchorage strengths provided in Division 88 (and all other related documents) have been obtained from tests on three-wythe walls, and that similar anchorage tests have never

Fig. 15. Localized veneer failure on west wall and southwest corner of a retrofitted building which survived the Northridge earthquake without structural damage, Santa Monica Boulevard: (*a*) global view of peeled veneer; (*b*) closeup view of veneer failure and poor collar joint; (*c*) damage to corner.



been conducted in two-wythe walls. While some engineers believe otherwise, there are effectively no experimental results to substantiate either position.

Non-representative push-test results

It is somewhat unfortunate that the mortars used in unreinforced masonry construction at the turn of the century in southern California were of generally poor quality. For a long time, mortar was made with large quantities of lime and readily available unwashed beach sand, with detrimental effects on the durability and strength of these mortars. Building code requirements were progressively tightened, up to the 1930s, to require increasing proportions of cement and the use of "clean sharp sand" (Alesch and Petak 1986).

Damage to numerous masonry buildings throughout North America, and worldwide, has clearly established that the seismic performance of unreinforced masonry buildings does not solely depend on mortar quality, but the low strength of masonry in southern California made difficult, sometimes impossible, the extraction of a masonry core for traditional laboratory testing and acquisition of material properties (Asakura 1987). Moreover, coring is a relatively damaging and expensive process, as a coring machine must be anchored to the wall, water is used, and the hole left in the wall is difficult to repair (Schmid 1981).

The push test has been developed to circumvent these difficulties. Given the large number of unreinforced masonry buildings retrofitted in the Los Angeles area in recent years,

Fig. 16. Out-of-plane failure of top-story two-wythe brick unreinforced masonry wall and one-wythe veneer of a retrofitted building in Hollywood, due to excessive height-to-width ratio.

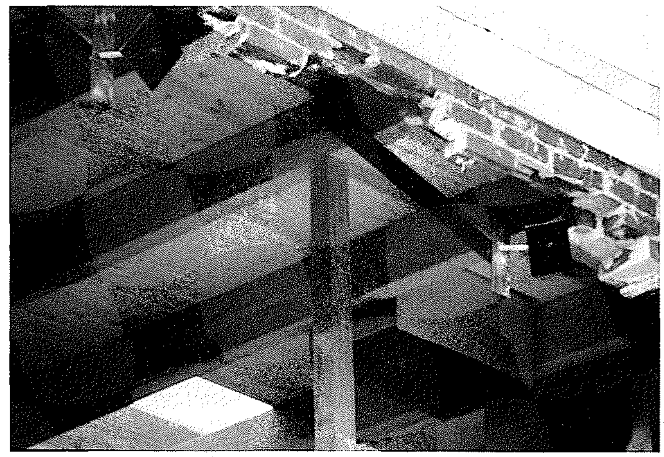


Fig. 17. Intermediate brace exposed by out-of-plane failure of unreinforced masonry wall of a retrofitted building on Santa Monica Boulevard and 4th Street, Santa Monica; vertical wood support visible under the diaphragm brace-end anchorage, and absence of effective vertical steel member along wall to carry the vertical force component of the steel brace.

this relatively simple test procedure has become a standard service provided by various laboratories. It requires the removal of a single brick unit to insert, in the resulting cavity, a small hydraulic ram. This ram is used to push on an adjacent brick whose head joint, opposite to the loaded end of the brick, has been removed. The load is thus applied horizontally, in the plane of the wythe, until either a crack can be seen or slip occurs. The resistance obtained from these push tests is used to determine the unreinforced masonry shear strength of the existing walls. Details of this procedure are available elsewhere (Clause A5 CGSEEB, Section A106.(c).3.A. of UCBC).

In view of the significant damage to retrofitted unreinforced masonry buildings during the Northridge earthquake, questions have been raised regarding the reliability of some testing agencies. In some instances, tests have yielded surprising results, four to five times higher than expected for the typically poor quality mortars of southern California, and retesting of some retrofitted buildings damaged by the earthquake illogically yielded higher results than originally obtained prior to the earthquake.

Although the structural engineer is responsible for determining on-site the exact test locations, this decision has apparently been delegated, over the years, to the testing agencies. As a result, some agencies have adopted a bad habit of making all tests at eye level for simplicity, thus typically missing the locations where deterioration has occurred and lower values for the masonry strength would be obtained. Furthermore, it has been reported that some agencies conduct tests too rapidly, which can produce significant variations in results. Subsequently to these findings, the City of Los Angeles is contemplating requiring that push tests can only be conducted by technicians trained and licensed by the City.

This points to the need to perform a large number of *in situ* push tests on each wall, at various locations across the walls, to properly investigate the quality of the masonry throughout the building. Even though it is always probabilistically possible to miss a local deficiency, the evaluation procedure partly takes into account this large variability of masonry strength.

Incomplete intermediate wall bracing system

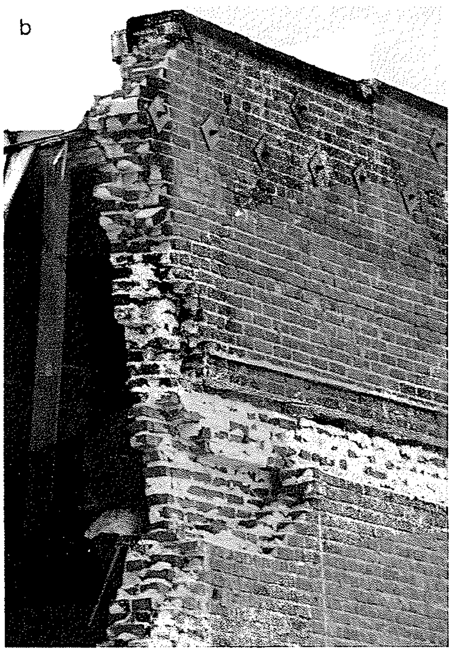
Walls found to have a height-to-thickness ratio (h/t) exceeding prescribed limits can be laterally supported by continuous bracing members, or the wall height can be reduced by bracing the wall using intermediate bracing element supports connected to the floor or roof (Section A110.(c) of the UCBC, Clause A6 of the CGSEEB). When the latter is chosen, special care must be taken to (*i*) ensure that the floor or roof framing stiffness is sufficiently large to avoid having the bracing act as a knee-brace for the live loads; (*ii*) provide sufficient anchorage resistance at both ends, considering all force components; and (*iii*) install the brace at a small sloping angle between the brace and horizontal diaphragm to which it connects, to minimize horizontal movements at the wall brace-end due to vertical movements at the diaphragm brace-end.

Since brace members can only carry axial forces, there are divergent opinions within the structural engineering community of southern California as to whether intermediate braces can be effective, unless the wall anchors at the end of the brace are properly designed to be able to resist both the shear and tension force components of the brace, or unless steel truss members are designed and added along the masonry wall to carry the vertical component of the diagonal brace member. Only in the latter case would tension-only wall anchors be acceptable. In Fig. 17, a vertical wood support can be seen immediately under the diaphragm brace-end

Fig. 18. Out-of-plane wall failure due to unbraced ceilings battering the walls, and inadequate anchorage from intermediate braces at ceiling level, for retrofitted building on Lankershim Boulevard, in North Hollywood: (*a*) global view of failed wall; (*b*) closeup view of roof anchorage and wall construction.

a

b



anchorage to practically eliminate the vertical deflections there, but a vertical steel member along the wall has not been provided to carry the vertical force component of the steel brace.

Flexible unanchored ceiling system pounding walls below roof line

Even when walls are anchored at all floors and roof levels, heavy ceilings hanging from the roof without proper cross-bracing can act as a battering ram against the walls. An example of this problem is illustrated in Fig. 18. From roof to base, the height-to-thickness ratio of this wall clearly exceeds the permissible limits, and the sparse government anchors at the ceiling level, as well as some intermediate braces installed at a very steep angle, proved ineffective in preventing collapse.

Fig. 19. Out-of-plane wall failure due to ineffective intermediate braces and battering action of unbraced ceilings, for retrofitted building located on Washington Street, Culver City.

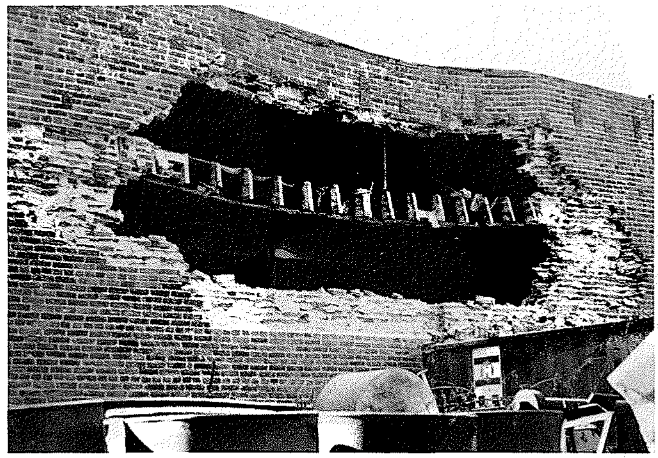
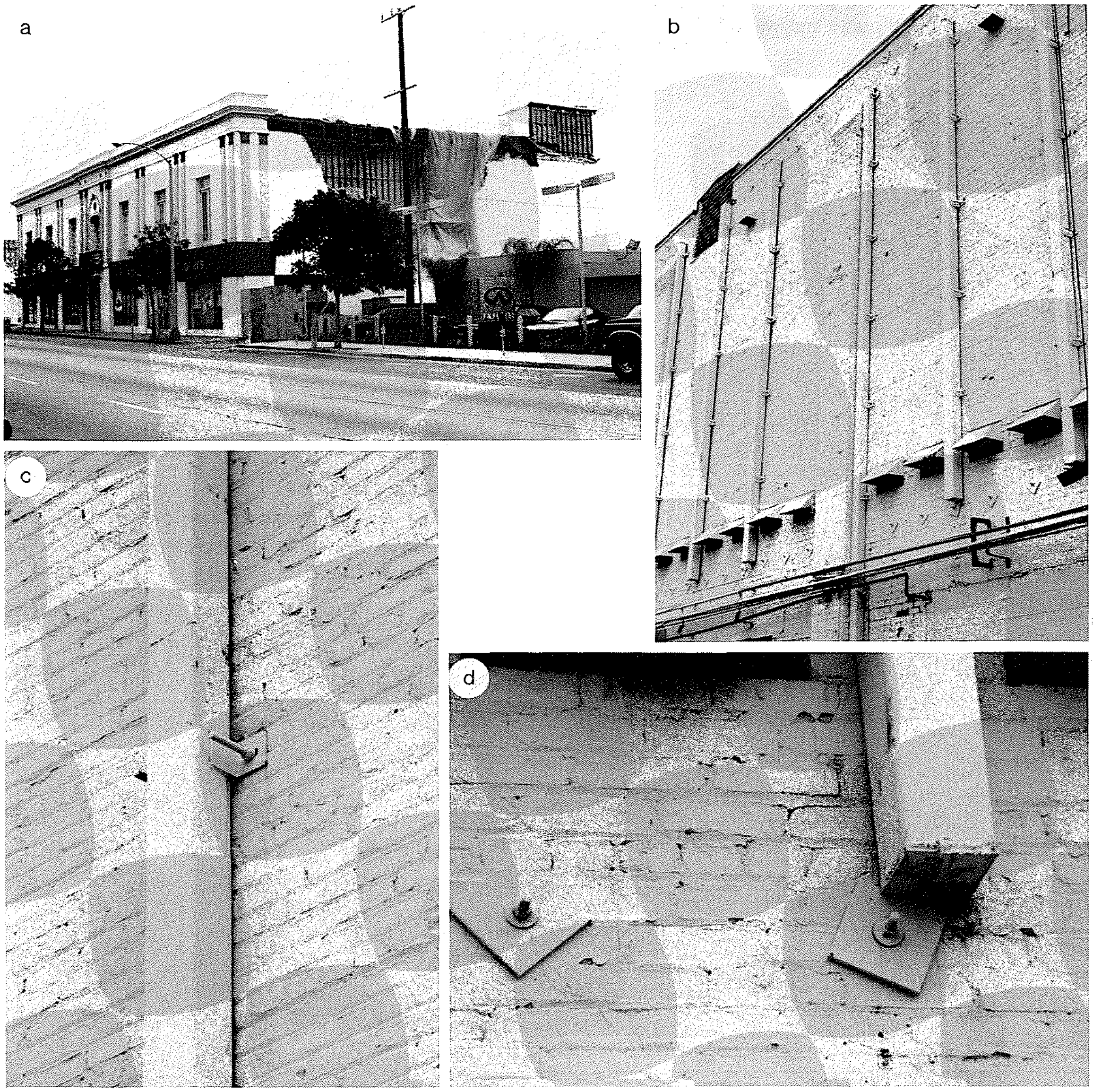


Fig. 20. Out-of-plane failure and near collapse of the central taller portion of a parapet, due to an incomplete retrofit, Washington Street, Culver City.

Another example of this undesirable behaviour is shown in Fig. 19. Again, the original government anchors and newer intermediate braces, visible at 2.25 m (7.5 ft) spacing at the ceiling level, proved ineffective against the battering action of the ceiling on this 330 mm (13 in.) wall. It is noteworthy that maximum spacing of intermediate braces is restricted to 1.8 m (6 ft) by the UCBC (Section A110.(e).3.), or 6 wall thicknesses by the CGSEEB (Clause A6). Moreover, the anchor plates of the intermediate braces visibly have approximately only 70% of the minimum area of 20 000 mm^2 (30 $in.^2$) required by the UCBC (Section A110.(a).2.) and CGSEEB (Clause A7) for tension anchors in walls at least three-wythe thick.

Constructing a plywood wall in the ceiling space whenever the roof joists and ceiling rafters line up would be an effective and simple retrofit to avoid the problem of pounding ceilings. Moreover, ceilings with substantial mass should be braced to the roof (or floor) diaphragms along their perimeter, and ceilings with substantial rigidity should be

Fig. 21. Santa Monica Masonic Temple: (*a*) out-of-plane failure of slender unreinforced masonry west walls with excessive height-to-thickness ratio; (*b*) undamaged south wall retrofitted with vertical bracing members; (*c*) closeup view of typical brace intermediate anchors; (*d*) closeup view of typical brace base detail.



anchored with tension bolts spaced at no more than 1.8 m (6 ft) in accordance with the UCBC (Section A110.(a).1.) and CGSEEB (Clause A7).

Incomplete or partial retrofits only

Incomplete retrofits can only be expected to provide, at best, satisfactory seismic behaviour of the properly strengthened parts of the building. For example, although parapet braces were added at 1.8 m (6 ft) spacing throughout the building shown in Fig. 20, additional braces would have been needed to restrain the taller central portion of the parapet, which rocked severely and almost collapsed during the Northridge earthquake.

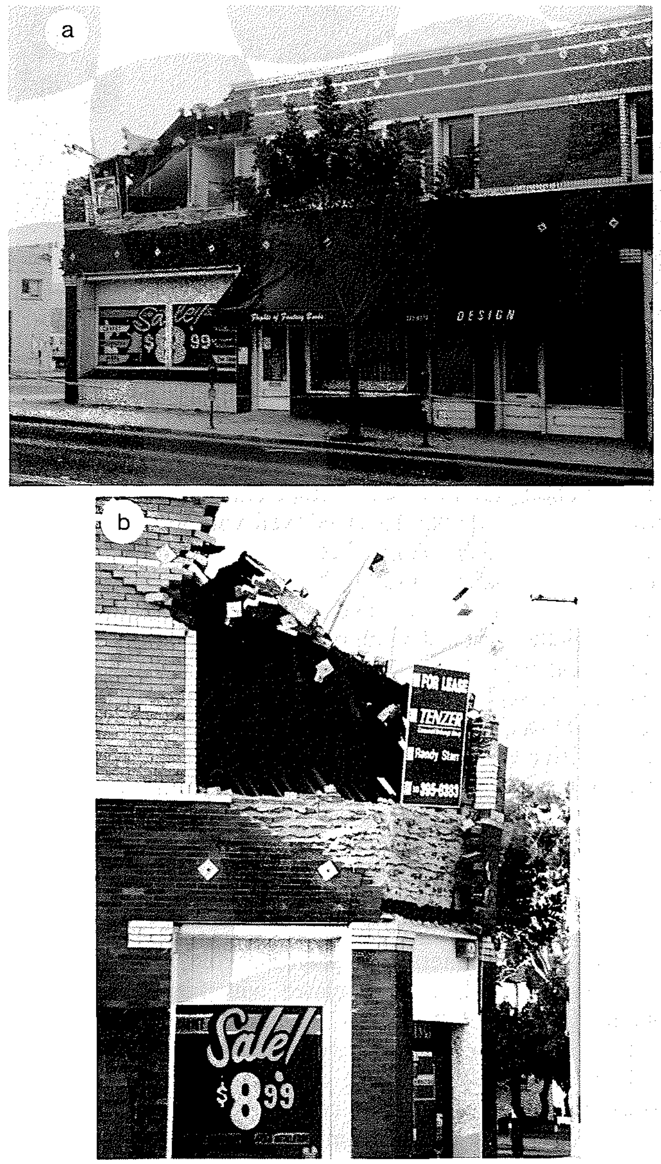
In many instances, failures typically occurred in slender unreinforced masonry walls with excessive height-to-thickness ratios where the required additional vertical bracing members (Section A110.(e).2. of the UCBC, Clause A6 of the CGSEEB) were missing. In one striking example, part of the wall of a grocery store in Hollywood collapsed in an out-of-plane manner exactly where a single brace was missing. In another example, a considerable portion of the unbraced west wall of the Masonic Temple in Santa Monica failed in an out-of-plane manner, whereas its properly braced south wall survived undamaged (Fig. 21). Although the south wall could have been rated as less aesthetically pleasing before the earthquake, it is clearly not the eyesore the west wall has become following the earthquake.

Behaviours not explicitly addressed by existing seismic evaluation procedure requirements

Severe damage to corners of seismically retrofitted buildings also occurred in numerous instances (e.g., Figs. 22–25).

Fig. 22. Severe corner damage in retrofitted building: collapsed corner wall L-shaped in plan, Hollywood Boulevard, Hollywood.

Fig. 23. Severe corner damage in retrofitted building: collapsed small piers which were located between openings and seismically excited along both of the building's principal directions, Santa Monica Boulevard, Santa Monica: (*a*) global view of east corner of building; (*b*) closeup view of corner damage, including veneer damage.



This is particularly significant, since, in many cases, the retrofitted buildings were otherwise undamaged. Numerous factors contributed to this damage:

- Flexible wood floor diaphragms in existing unreinforced masonry buildings behave as deep beams spanning between end walls, and the tendency for in-plane rotation of these diaphragms' ends can induce damage at the walls' corners.
- Although unreinforced masonry buildings are typically seismically rehabilitated with new added structural elements to provide shear transfer between diaphragms and reaction walls, it has been alleged that there may be some looseness in these connections which may allow the diaphragms to slide slightly and impact the unreinforced masonry walls in the transverse direction (Erikson 1981).
- Of considerable importance, the existing seismic evaluation procedures treat walls as plane structural elements, which is obviously an inaccurate modelling at corners where walls and piers L-shaped in plan can be significantly stiffer and behave quite differently than assumed (Fig. 22). For example, corner piers assessed as capable of rocking can actually be restrained from doing so by their orthogonal leg, and fail instead under horizontal or vertical shear at the corner.
- Some corner piers are actually small structural elements between openings, and are simultaneously seismically excited along both of the building's principal directions (Figs. 23 and 24). Such bidirectional in-plane effects are neglected by the existing seismic evaluation procedures.
- In buildings having some nonorthogonal walls, the corner response where the walls meet nonorthogonally is not well understood (Fig. 25). The diaphragm is considerably less flexible in its acute corner, and may need to be anchored for larger forces.

It is noteworthy that the special procedure adopted in the Los Angeles region for the seismic rehabilitation of existing unreinforced masonry buildings does not account either for the increased vulnerability of unreinforced masonry walls due to the combined action of in-plane and out-of-plane dynamic excitations (Bruneau 1994*a*). Indeed, although the special procedure checks the dynamic stability and strength of walls and piers in both the in-plane and out-of-plane directions, it is done using separate empirical equations derived from tests conducted on individual walls and piers only subjected to a single direction of dynamic excitation.

The combined in-plane and out-of-plane interaction was visible in many of the damaged retrofitted buildings (e.g., Fig. 26). These unfavourable conditions may have been worsened in some buildings by the presence of open-fronts on more than one side of the first story. Ductile moment-resisting frames typically introduced in these open-fronts can, contrary to walls on uplifting foundations, amplify ground motions to produce more severe dynamic excitations at the edges of the floor diaphragms. For example, the build-

Fig. 24. Severe corner damage in retrofitted building: collapsed small pier which was located between openings and seismically excited along both of the building's principal directions, Hollywood Boulevard, Hollywood.

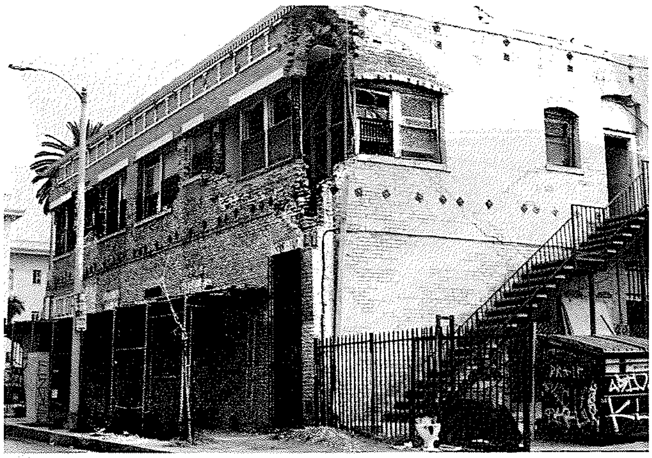
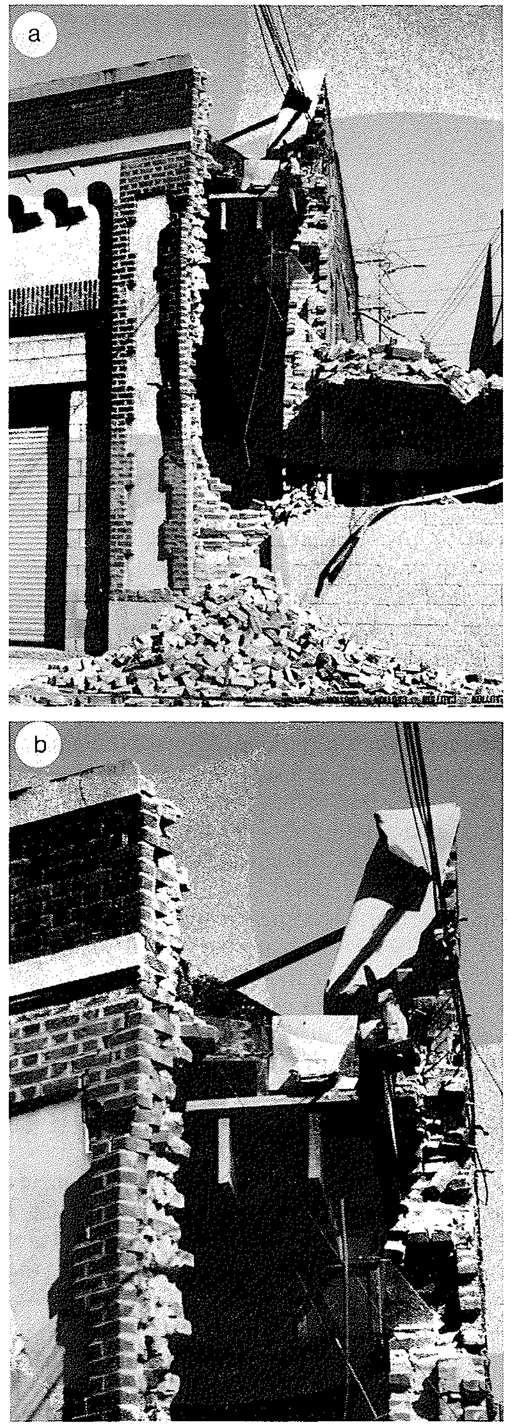


Fig. 25. Severe corner damage in retrofitted building at the acute corner of walls meeting nonorthogonally, Washington Street, Culver City: (*a*) global view; (*b*) close-up at roof level.



ing shown in Fig. 26, and for which structural seismic rehabilitation has just been completed at the time of the Northridge earthquake, survived well with the exception of the east wall which collapsed at the second story. This two-story building (45.8 m (150 ft) by 26.8 m (88 ft) in plan) had been reinforced with two and three moment resisting steel frames on the west and south first-story open facades respectively, and its 225 mm (9 in.) thick unreinforced masonry walls at the second floor had been shotcreted on the street sides and heavily anchored on the alley and back sides. Evidence of five very old infilled openings could be seen at each story of the east alley-side wall, which may explain the presence of a steel beam embedded in the masonry at the floor level, and which has likely negatively affected the out-of-plane dynamic stability of this wall. Severe in-plane cracking was visible on the remaining uncollapsed portions of the second-story wall, which suggests that combined in-plane and out-of-plane interaction played a role in this failure. Undesirable behaviours associated with intermediate wall bracing and two-wythe walls, reviewed earlier, have also likely played a role in this particular failure.

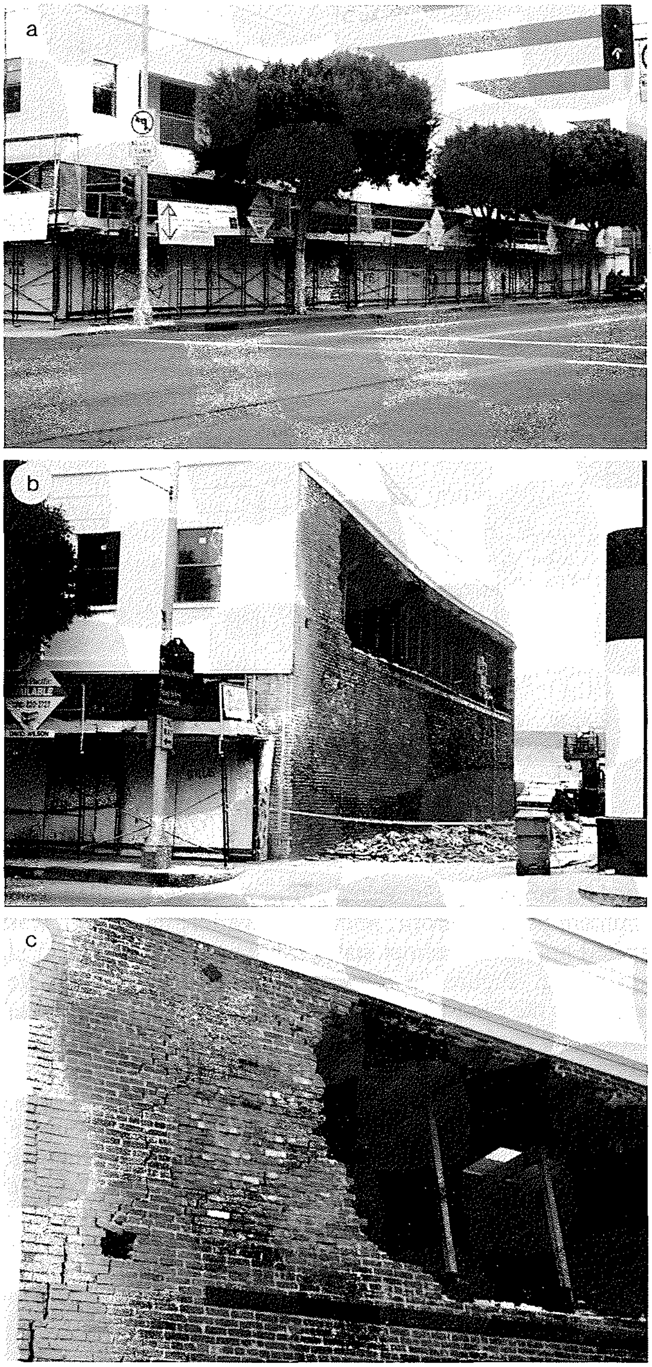
Explicit requirements and guidelines to prevent corner damage, and address possible combined in-plane and out-of-plane failures, are absent from the existing codified procedures for the seismic rehabilitation of unreinforced masonry buildings. Special ties anchored at the corners and spanning the entire length and width of buildings might be a solution to this problem. Clearly, these aspects of seismic behaviour will have to be researched.

Finally, the observation of in-plane damage in retrofitted buildings should not be construed as a shortcoming of the existing seismic evaluation and rehabilitation procedures. As mentioned earlier, the intent of these procedures has always been to minimize the life-safety risk, not to prevent structural damage. Consequently, limited in-plane damage which poses no threat to life is tolerated, such as in-plane X-cracking of walls or piers (Fig. 27). However, it is somewhat unsettling that such severe in-plane cracking occurred at a level of effective ground excitations below the design-basis level implicitly considered by the retrofit ordinance.

Historical buildings

As observed in past recent earthquakes, and particularly after the Loma Prieta (San Francisco) earthquake of 1989, North American historical structures of masonry construction are most seismically vulnerable (Merritt 1990; Bruneau 1991; Cross and Jones 1991; Kariotis et al. 1991). It is paradoxical, and odd, that in the past, the historical preservation designa-

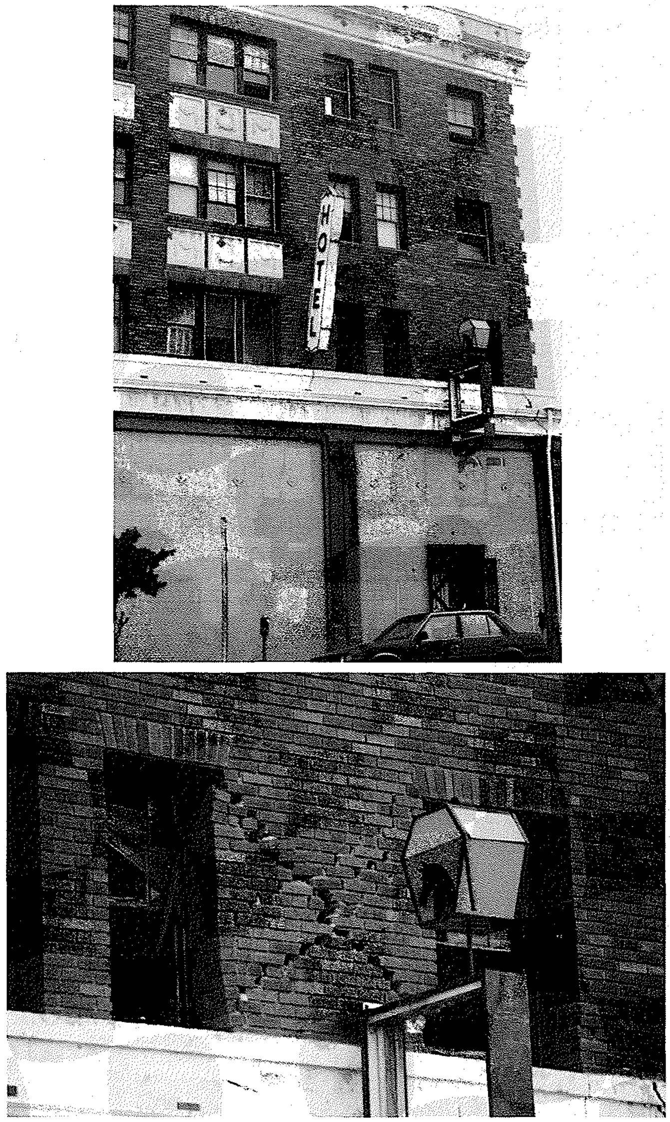
Fig. 26. Failure partly due to combined in-plane and out-of-plane interaction, retrofitted unreinforced masonry building, Santa Monica Boulevard, Santa Monica: (*a*) global view of first-story open fronts (during construction); (*b*) out-of-plane collapsed east wall at second story; (*c*) severe in-plane cracking visible on remaining uncollapsed portions of the second-story wall.



tion of a building has sometimes been used as an excuse to delay seismic rehabilitation work. While a comprehensive overview of damage to historical structures is beyond the scope of this work, two examples are noteworthy.

The First United Church of Santa Monica was an unreinforced masonry building built in 1924, to which some para-

Fig. 27. Damaged retrofitted Staton Hotel, Western Avenue, Hollywood: (*a*) global view; (*b*) severe in-plane X-cracking of piers.



pet bracing was added, in 1975, along the west wall and above the entrance stairs, to protect a shorter neighbouring building and the main entrance way. More recently, a structural engineer retained by the parish, to comply with the Santa Monica Ordinance, found the existing building to be seismically deficient, and developed a complete seismic retrofit strategy, using (*i*) comprehensive anchorage of floors and roofs; (*ii*) bracing of all parapets where needed; (*iii*) replacement of existing substandard anchor plates where needed; (*iv*) center core drilling in the middle of the 325 mm (13 in.) walls to reinforce the tall and potentially dynamically unstable walls in their out-of-plane direction, while preserving the historic fabric of the building; and (*v*) addition of wood crosswalls as per the UCBC requirements. As this church served a community of approximately 1000, it was not possible to raise the necessary funds to perform this seismic strengthening and a financial hardship exemption was requested. However, as some gravity-load-resisting structural elements were

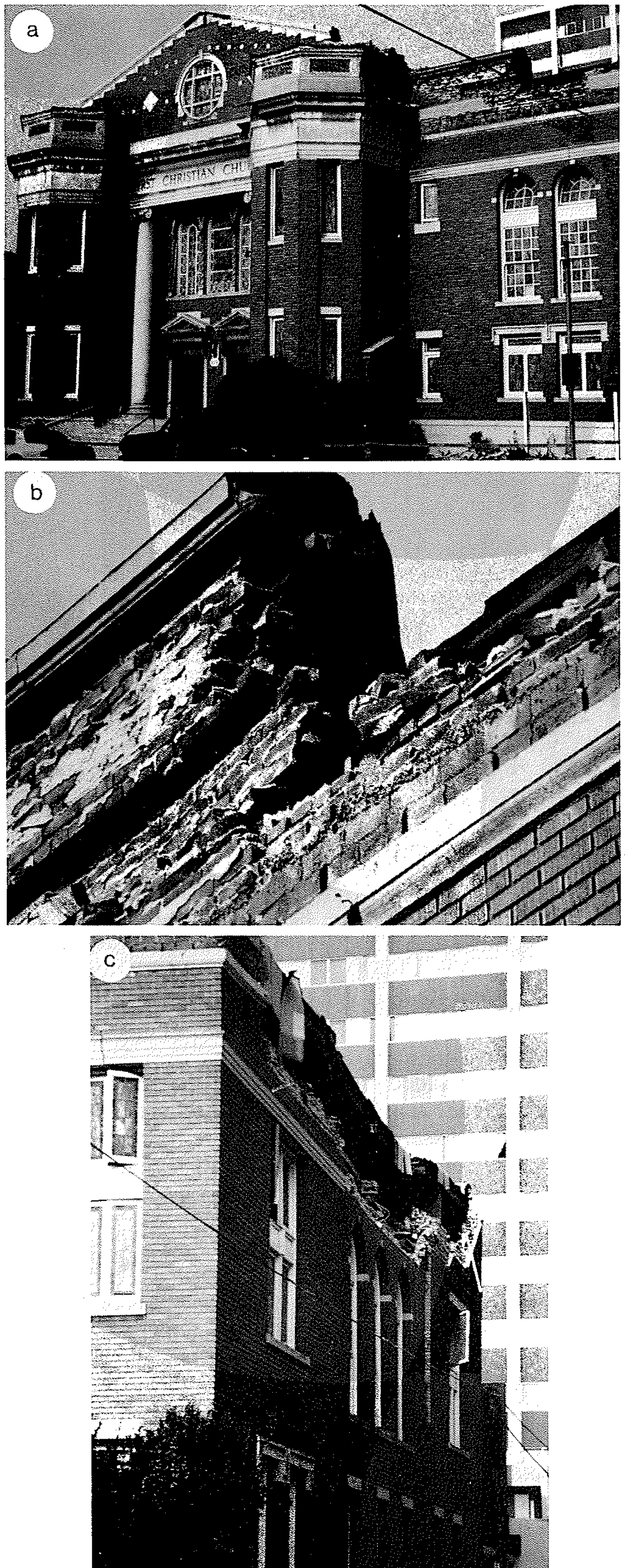


Fig. 28. Damage to First United Church of Santa Monica: (*a*) global view, showing in-plane cracking of turrets and collapse of unbraced parapet; (*b*) close-up of collapsed unbraced parapet, and poor quality collar joint; (*c*) out-of-plane collapse of top of east wall.

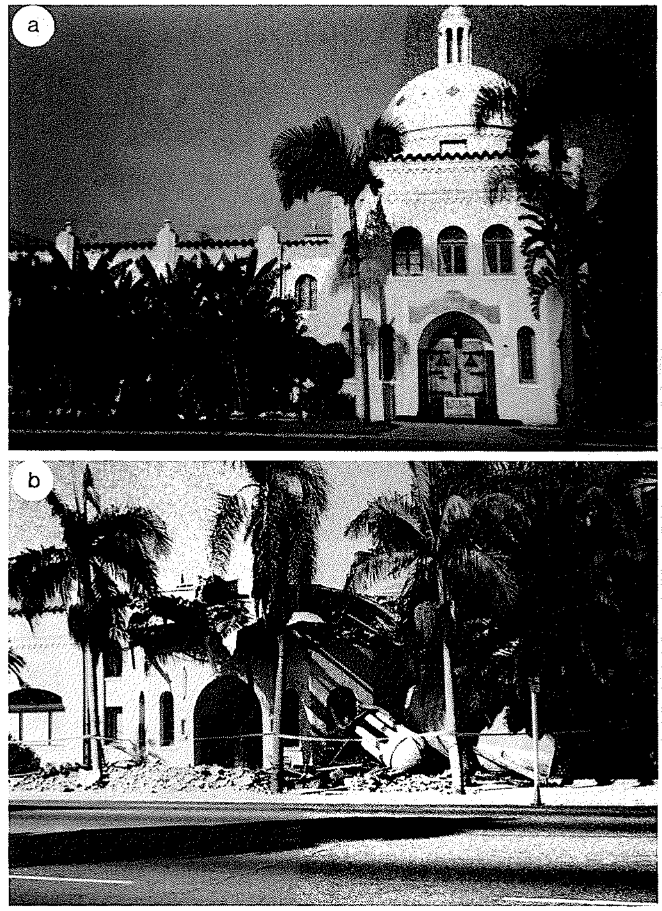
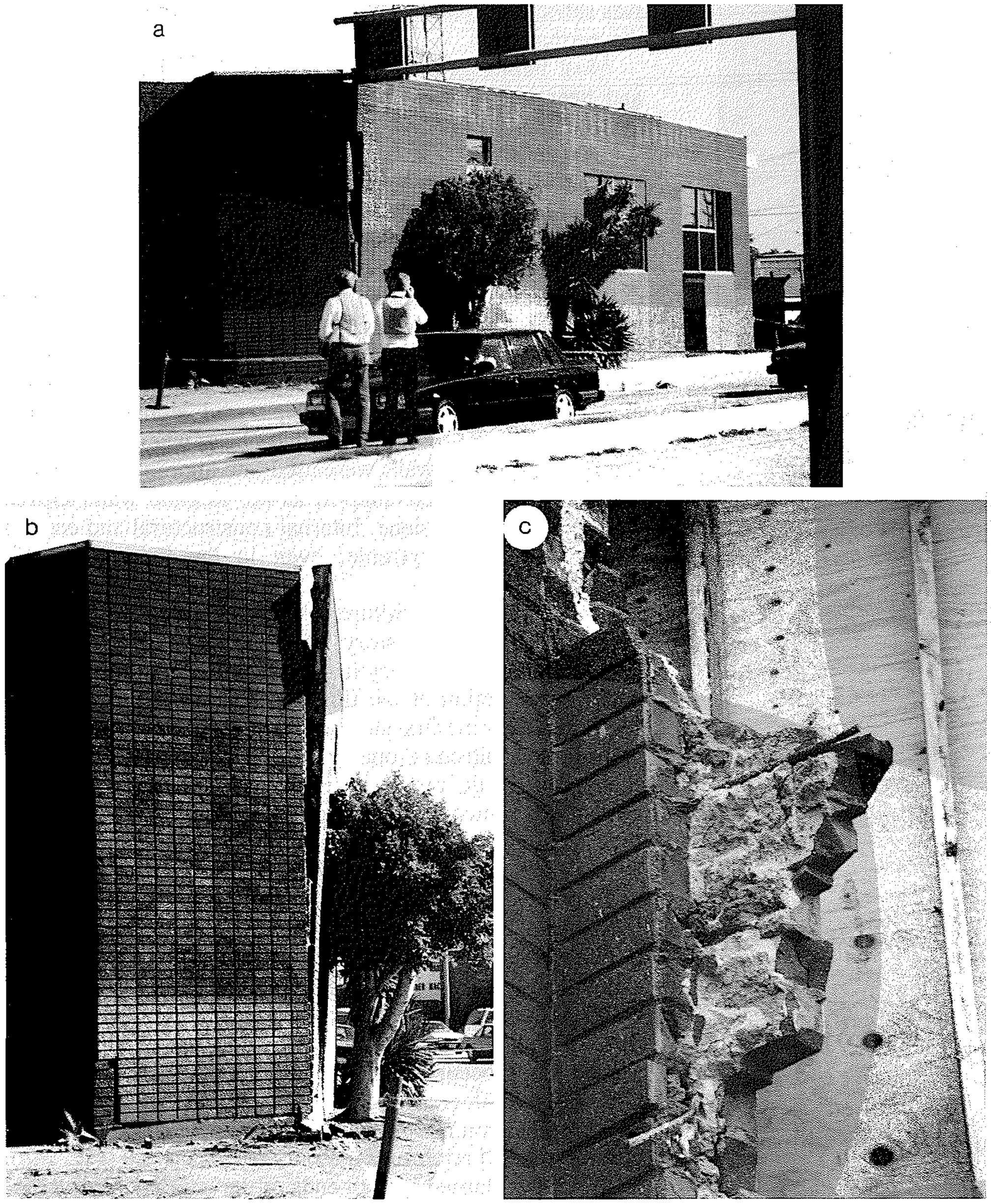


Fig. 29. Damage to Metropolitan Community Church of Los Angeles, in Culver City: (*a*) front view, showing tower and Byzantine-style dome — photo taken prior to the earthquake; (*b*) collapsed tower, after the Northridge earthquake.

found to be poorly connected, crosswalls were added as a minimum partial retrofit. Shortly thereafter, the church was granted unlimited exempt status on account of its historical importance. Figure 28 illustrates some of the extensive damage to the First United Church of Santa Monica from the Northridge earthquake: considerable parts of the north and east walls collapsed, a large portion of the veneer on the west walls peeled off, severe in-plane cracking occurred in turrets, unbraced parapets on the facade collapsed, and some of the aforementioned questionable gravity-resisting structural elements toppled. Clearly, the added crosswalls prevented local collapse of the building, and the parapets braced in 1975 survived. However, the extent of damage made repairs prohibitive, and the church was demolished shortly after the earthquake.

The Metropolitan Community Church of Los Angeles, in Culver City, provides another example of an historic structure for which partial retrofit strategy proved insufficient to prevent collapse (Fig. 29). This building (roughly 30.5 m (100 ft) by 28 m (92 ft) in plan) had a 12.2 m (40 ft) tall octagonal tower at its southeast corner, topped by a Byzantine-style dome which won the building a landmark status. For architectural reasons, the solid walls of the octagonal towers were discontinued and supported on beams and 150 mm (6 in.) deep steel columns at the first story,

Fig. 30. Damage due to lack of integrity between adjacent panels of stacked bond reinforced brick, and between panels and roof diaphragm, Adams Street, Culver City: (*a*) global view of building; (*b*) separated and severely leaning panel; (*c*) close-up of reinforcement exposed after demolition of damaged part of wall.



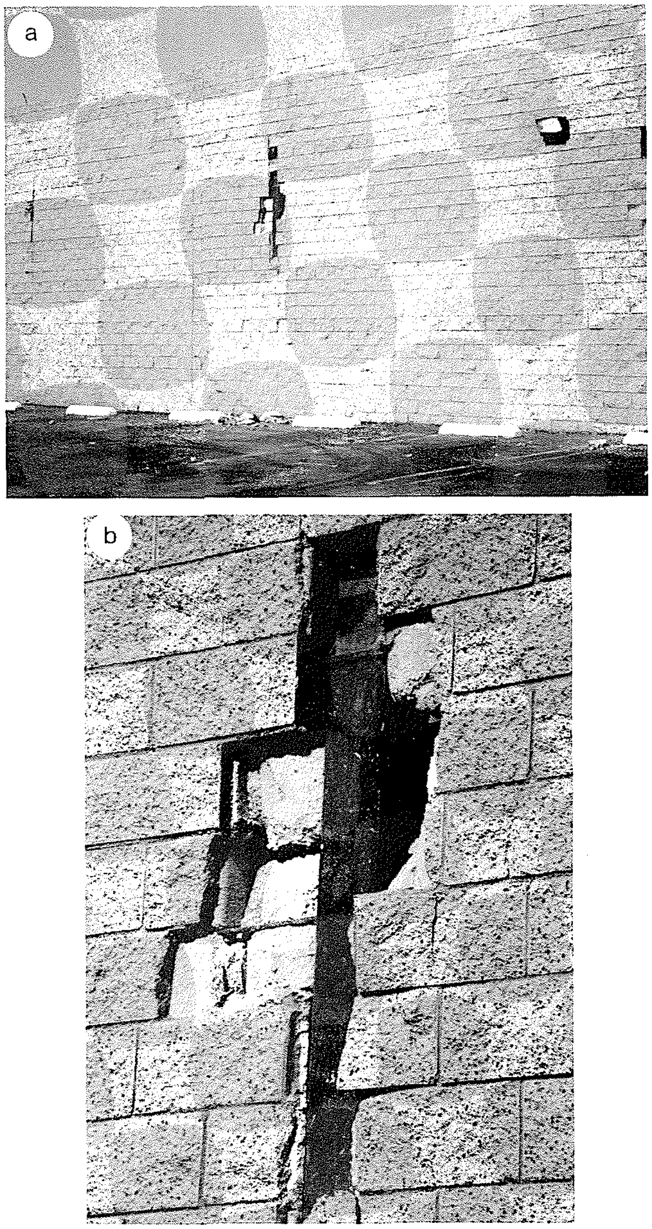
except for those walls visible from the outside. Seismic rehabilitation of the entire church was conducted even though this work proved a strenuous financial hardship on the small community served by this building. Retrofit of the tower consisted mainly in anchoring it to the adjacent floors of the main building. Clearly, in such a case, the standardized seismic strengthening procedures of the UCBC are not sufficient, and a thorough structural strengthening of that tower would have been necessary. Not surprisingly, the tower collapsed during the Northridge earthquake. The remaining more regular part of the structure survived.

Damage to buildings of such historical value and heritage beauty is particularly sad, but it emphasizes the necessity to perform serious seismic adequacy assessment of these existing buildings if their preservation is truly intended. The fear that structural engineering interventions will unduly alter the heritage character of a building weights little against the potential losses due to "a seismic intervention." It is important that historical preservation goals be formulated clearly to resolve this apparent dichotomy.

Reinforced masonry buildings

Buildings with reinforced masonry shear walls generally performed well throughout the Los Angeles area. This observation is in agreement with a recent report (TMS 1994) which has documented numerous examples of satisfactory seismic performance of reinforced masonry buildings located in the epicentral region. However, a few exceptions to this good behaviour have been observed by the author, and are reviewed in the following.

Fig. 31. Reinforced masonry wall damaged by pounding and punching of wood beams, Lamp Plus Building, Northridge: (*a*) global view of damaged wall; (*b*) closeup view.



Damage due to lack of integrity between adjacent panels of stacked bond reinforced brick, and between panels and roof diaphragm, was observed (Fig. 30). This emphasizes the need and importance of continuity reinforcement in masonry, even though the very light reinforcement present in the walls could preserve the integrity of the individual panels. It is noteworthy that panels with the least amount of openings, thus with the largest reactive mass, pulled apart more severely from the roof of that building.

Damage has been observed in many reinforced masonry commercial buildings of a shopping plaza on the southwest corner of the Tampa and Nordroff intersection in Northridge (i.e., a few hundred feet from the Northridge Fashion Center, on the northwest corner of the same intersection, which suffered a tremendous amount of damage and collapses, e.g., EERI 1994*a*; CAEE 1994; and other papers in this special issue of the Canadian Journal of Civil Engineering). Damage to some of these reinforced masonry buildings was sufficiently extensive to warrant demolition shortly after the earthquake.

Most buildings in this plaza had wood diaphragm roofs and beams supported internally on small steel posts only present to carry gravity loads, and connected to exterior reinforced masonry walls for lateral-load resistance. When medium-span heavy wood beams were used, steel columns were embedded in walls to help carry the gravity loads at the support. Clearly, the dynamic in-plane motion of the diaphragms produced pounding of the wood beams on the exterior walls, with beams punching through the reinforced masonry walls in some cases. When steel columns were embedded in the walls to carry gravity loads, damage to these wall did not trigger collapse (Fig. 31), whereas, in the absence of steel columns, collapse did sometimes occur (Fig. 32). In other buildings of that plaza, where tall slender reinforced masonry walls were used, the improperly anchored roofs slipped-off their support as the flexible walls moved significantly out-of-plane. Internal nonstructural and content damage was also extreme, even in the lesser damaged buildings.

Nonductile shear failure of reinforced masonry piers occurred at the first story of the Kinzey at Northridge Condominiums. This very large residential building was still under construction at the time of the earthquake. Damage concentrated at the first story which had a significantly larger number of architectural openings (doors, windows, and garage access) due to the presence of a parking garage, making for a relatively softer story. Moreover, at that level, the aspect ratio of most reinforced masonry elements ensured that failure would be of the nonductile shear type (Fig. 33).

Finally, it is noteworthy that the Northridge Kaiser Permanente Medical Center, a five-story reinforced concrete building which collapsed, had a very large and stiff reinforced brick masonry wall spanning the entire 20 m (65 ft) width of the building in the east–west direction. Damage to this building was considerable, as a soft-story failure of the second floor occurred further to excessive damage to the nonductile reinforced concrete frame, which formed a collapse mechanism and swayed in the north–south direction, pushing the stiff reinforced masonry walls outward (Fig. 34*a*). Walls thus collapsed at both ends of the building. Numerous casualties would have occurred there had the earthquake struck during working hours.

Observation of the walls' reinforcement details, also made visible by the collapse (Fig. 34*b*), confirms that concrete edge-columns were embedded in the reinforced brick masonry end-walls designed to behave structurally if excited in the east–west direction. These heavily reinforced and stiff walls unfortunately provided little contribution to structural response in the north–south failure direction. Plans for the seismic retrofit of this building were ready; a construction permit had been obtained two months earlier, and the contractor was about to begin the seismic rehabilitation work when the earthquake struck. Four new 300 mm (12 in.) thick reinforced concrete walls were to be shotcreted, each wall spanning 14.6 m (48 ft) in the north–south direction. No retrofit

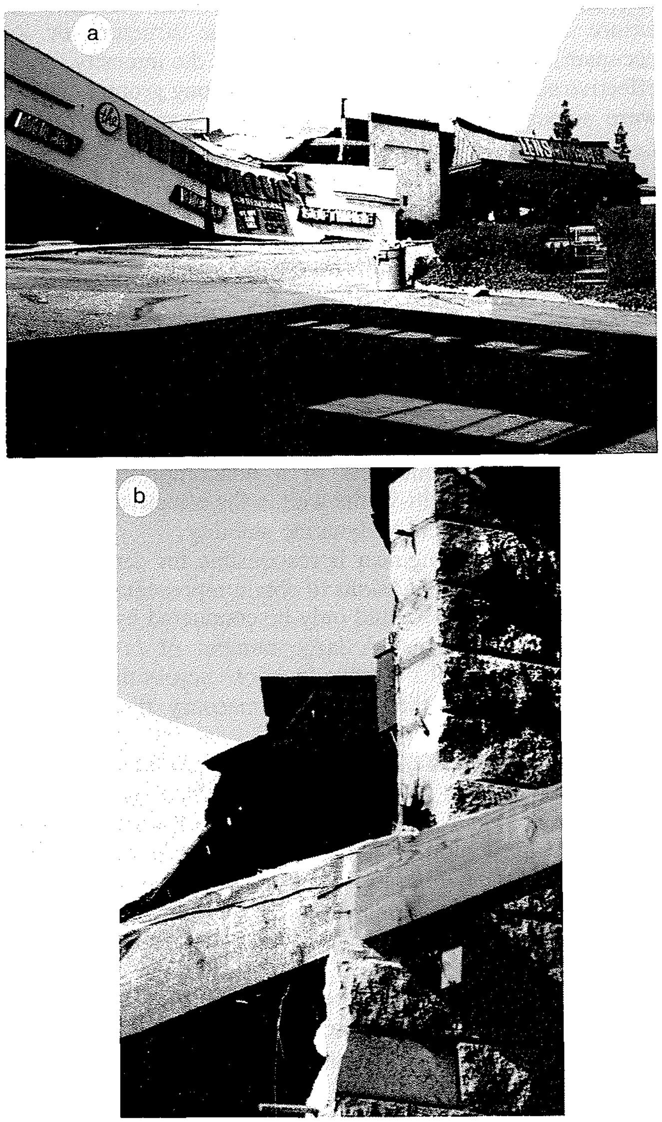


Fig. 32. Collapsed Warehouse and Lenscrafter building due to pounding and punching of beams through reinforced masonry wall, Northridge: (*a*) global view; (*b*) closeup view of wood beam punched through a reinforced masonry wall.

work was scheduled for strengthening the building in the east–west direction. This building would have undoubtedly survived the earthquake had this retrofitting work been completed prior to the earthquake.

Conclusions

This paper has provided an overview of the seismic performance of masonry buildings during the January 17, 1994, Northridge earthquake. Many unretrofitted and retrofitted masonry buildings, located within a radius of roughly 30 km from the epicentre, performed rather poorly during this earthquake. Numerous casualties and injuries could have resulted from the reported failures (including falling debris) had this earthquake not occurred in the middle of the night.

The seismic vulnerability of unreinforced masonry buildings has been reemphasized by this earthquake, particularly

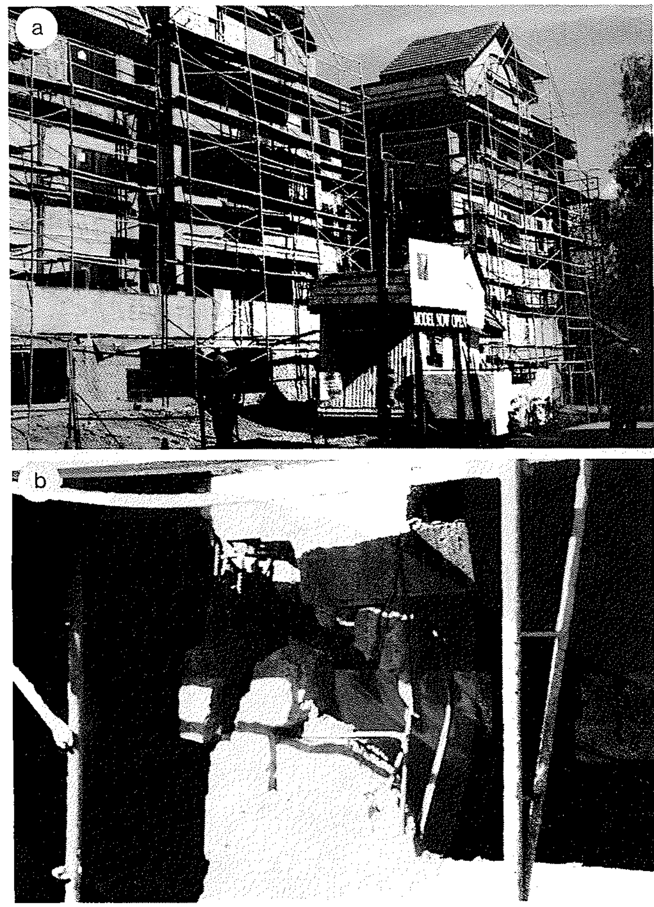
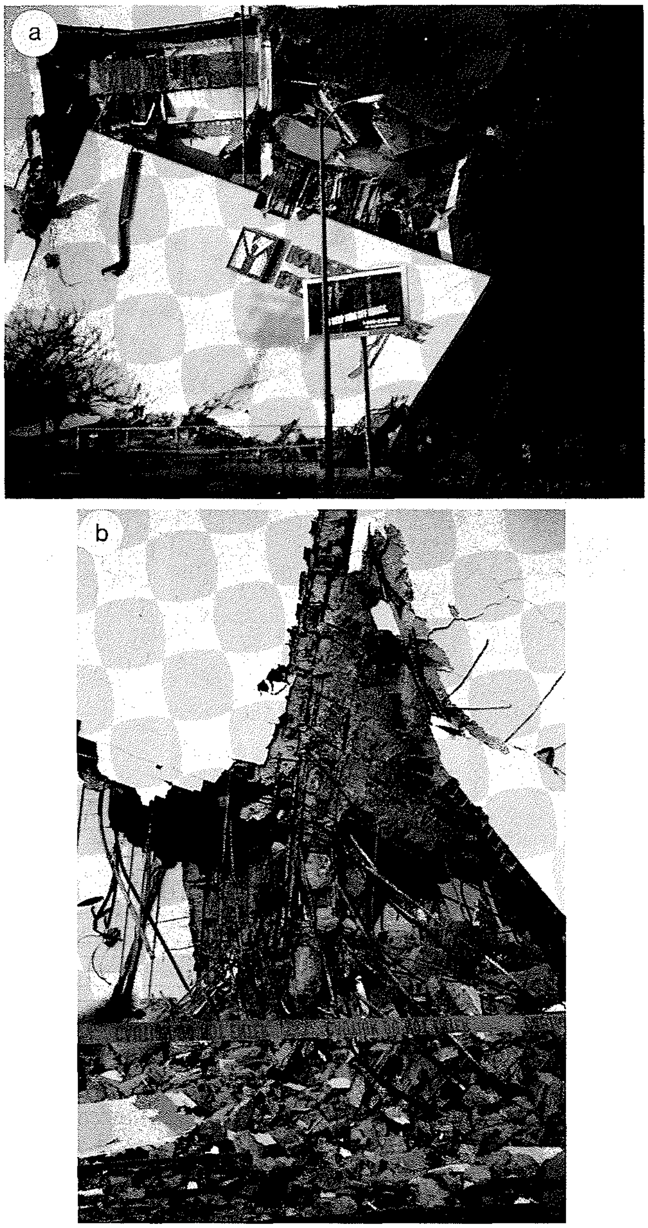


Fig. 33. Damaged reinforced masonry building, Kinzey at Northridge condominiums: (*a*) global view of building during construction; (*b*) closeup view of severe shear failure of first-story reinforced masonry pier.

in contrast with the generally good performance of comparable seismically retrofitted buildings. Based on the observations of damage from this earthquake reported in this paper, the following conclusions are possible:

- Damage to unreinforced masonry buildings, overall, was less extensive than what has been encountered after other Californian earthquakes of this severity. This can be attributed to the fact that the inventory of unreinforced masonry buildings in the near epicentral region had already been depleted by the 1971 San Fernando earthquake, and that urbanization in and around Northridge mostly occurred after the 1933 ban on unreinforced masonry construction. Such a ban in seismic regions, as also done by the NBCC, is clearly justified.
- In the downtown core of older cities farther from the epicentre but where peak effective acceleration exceeded $0.20g$, damage to unreinforced masonry buildings was considerable. As expected, out-of-plane failures of wall and parapets were the most common type of damage. In-plane failure and combined in-plane and out-of-plane failures were also observed.
- Existing wall anchors, of archaic types and layout, again proved ineffective in preventing out-of-plane failures.
- Nonstructural unreinforced masonry, particularly defi-

Fig. 34. Collapse of the Northridge Kaiser Permanente Medical Center, a five-story reinforced concrete building having solid stiff reinforced brick masonry wall spanning the short direction: (*a*) global view of collapse in the north–south direction; (*b*) closeup view of reinforcement details of end walls.



ciently anchored veneers, hollow clay-tile nonstructural infills not properly tied to their structure, chimneys, and property line walls, suffered severe damage during this earthquake. This emphasizes the need to also address the life-safety hazard posed by falling masonry debris from non-structural components in seismic rehabilitation activities.

- Some notable heritage unreinforced masonry structures lost in this earthquake could have survived had they been seismically rehabilitated.

Nonetheless, even though seismic rehabilitation activities have been generally successful in mitigating the seismic risks inherent to unreinforced masonry buildings, approximately 10% of all seismically retrofitted unreinforced masonry buildings in the Los Angeles area suffered structural damage, some of it life-threatening. While the Californian seismic-risk mitigation ordinances have never been intended to ensure absolute seismic survival of the structure or its occupant, this damage is significant since the intensity (peak effective accelerations) and duration of strong ground motion were somewhat less than the maximum expected for the Los Angeles area. With a notable few exceptions described in this paper, most of this damage can be attributed to short-comings in the application of the existing seismic rehabilitation procedure used in the Los Angeles area, and not to faults in the procedure itself. From the perspective of seismic structural rehabilitation work, the following lessons can be learned from the observed damage:

- The structural engineer must conduct a site visit of the building to be seismically rehabilitated. Wall thicknesses should not be taken from existing drawings which often only show total wall thicknesses, since it is a mistake to include, when present, the thickness of a noncomposite exterior veneer wythe of a multi-wythe wall in the effective thickness used in the calculation of dynamic stability.
- The structural engineer is responsible for determining, on-site, the exact test locations to obtain representative push-test results. Push test should only be conducted by qualified and trained technicians. A large number of *in situ* tests should be performed at various locations across the walls to properly investigate the quality of masonry throughout a building.
- Special care must be taken to inspect whether an adequate collar joint is present during individual *in situ* push tests done to obtain material properties. All areas of poor workmanship should be identified, and masonry not meeting the specified standards must be removed and replaced.
- Veneer anchors designed as per current requirements must be installed if the existing conditions are found to be deficient.
- Intermediate braces can be effective if the wall anchors at the end of the brace are properly designed to be able to resist both the shear and tension force components of the brace, or if steel truss members are designed and added along the masonry wall to carry the vertical component of the force in the diagonal brace member.
- Ceilings with substantial mass should be braced to the roof (or floor) diaphragms along their perimeter, and ceilings with substantial rigidity should be anchored with tension bolts.
- Incomplete retrofits can only be expected to provide, at best, satisfactory seismic behaviour of the properly strengthened parts of the building.
- Explicit requirements and guidelines to prevent corner damage, and address possible combined in-plane and out-of-plane failures, are absent from the existing codified procedures for the seismic rehabilitation of unreinforced masonry buildings. These aspects of seismic behaviour have to be researched.
- Limited in-plane damage which poses no threat to life can still occur for buildings seismically rehabilitated according to existing procedures. According to the stated rehabilitation philosophy and objectives of these various codes and

guidelines, this type of damage is tolerable.

In spite of some spectacular failures, damaged reinforced masonry buildings were few, even in the near epicentral region. Poor anchorage at roof level, inadequate continuity of reinforcement, and the shear vulnerability of short piers were the main causes for the observed failures. Thus, buildings with large reinforced masonry shear walls performed generally well throughout the Los Angeles area.

The lessons learned from this important Californian earthquake provide an excellent opportunity for Canadian structural engineers to not only review their existing seismic structural rehabilitation practice, but also educate their clients about the liability ensuing from the lethal potential of a seismically deficient masonry building, particularly since knowledge now exists to economically mitigate these life-threatening hazards. It also provides Canadian heritage preservation agencies with an additional impetus to initiate a comprehensive program to remedy the potentially deficient seismic resistance of historic unreinforced masonry buildings, to ensure heritage preservation beyond future earthquakes.

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