Seismic evaluation of unreinforced masonry buildings — a state-of-the-art report

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The potential vulnerability of old unreinforced masonry buildings, designed with little or no consideration for seismic-design requirements, is well documented. In regions without seismic retrofit ordinances prescribing a specific method to evaluate existing unreinforced masonry buildings, engineers have generally resorted to either conservative methods or various advanced analytical models. Although some approaches have received broader acceptance than others, there is still no consensus among practising engineers in North America. To provide perspective on the spectrum of strategies available and a clear overview of the state-of-the-art on this topic, this paper (*i*) presents the theoretical background and practical applications of a new procedure to evaluate unreinforced masonry bearing wall buildings, developed in California and recently integrated into the new Canadian Guidelines for the Seismic Evaluation of Existing Buildings, and (*ii*) summarizes the findings from other recent experimental and analytical research activities on the seismic behaviour of unreinforced masonry buildings, and from advances in their modelling.

Key words: unreinforced masonry, masonry, earthquake, seismic response, state-of-the-art, evaluation, rehabilitation, analysis, models, buildings.

La vulnérabilité latente des vieux bâtiments de maçonnerie non armée, conçus sans tenir compte de réhabilitation sismique des exigences de calcul parasismique, est bien connue. Dans les régions où il n'existe aucune réglementation comportant une méthode précise d'évaluation des bâtiments de maçonnerie non armée, les ingénieurs doivent recourir à des méthodes prudentes ou à des modèles analytiques d'avant-garde. Bien que certaines approches soient plus acceptées que d'autres, il n'existe toujours pas de consensus parmi les ingénieurs en Amérique du Nord. Cet article traite du fondement théorique ainsi que des applications pratiques d'une nouvelle méthode d'évaluation des bâtiments de maçonnerie non armée avec murs porteurs, développée en Californie et intégrée récemment aux Lignes directrices visant l'évaluation sismique des bâtiments existants. Il résume également les résultats des plus récentes activités de recherches analytiques ou expérimentales sur le comportement sismique de bâtiments de maçonnerie non armée ainsi que des progrès en termes de modélisation. Il permet de faire le point sur l'état actuel des connaissances dans le domaine et de présenter la gamme de stratégies existantes.

Mots clés : maçonnerie non armée, maçonnerie, séisme, réponse sismique, état des connaissances, évaluation, réhabilitation, analyse, modèles, édifices.

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1. Introduction

The potential vulnerability of old unreinforced masonry buildings, designed with little or no consideration for seismic-design requirements, is well documented. Since this type of construction is prevalent in the downtown core of most North American cities exposed to seismic risk, where the population densities are usually the highest, the magnitude of this hazard can be appreciated. In regions of Canada where seismic-resistant design requirements have already been in effect for a few decades, some owners are slowly beginning to recognize this seismic hazard; the devastating effects of recent North American earthquakes on older unreinforced masonry buildings, in contrast to buildings designed to modern standards, have undoubtedly increased awareness of the problem. This has led, in some instances, to requests for the evaluation of the seismic-resistance adequacy of existing unreinforced masonry structures, the first step of any effective seismic hazard mitigation strategy.

While there is evidence that unreinforced masonry buildings can survive major earthquakes, the conditions required for satisfactory performance are not fully understood and the usual modern analytical tools are often unable to discriminate appropriately. Structural engineers retained to investigate the seismic resistance of various unreinforced masonry facilities promptly discover the limitations of current masonry design standards whose simplistic design guidelines are of little, if any, assistance in realistically assessing this resistance. Recent codes, such as the 1991 edition of the Uniform Code for Building Conservation (UCBC) (ICBO 1991*a*), the NEHRP Handbook for Seismic Evaluation of Existing Buildings (FEMA 1992*a*), and the Canadian Guidelines for Seismic Evaluation of Existing Buildings (CGSEEB) (NRC 1992), all inspired by a special procedure commonly known as the ABK-methodology (ABK 1984), are notable exceptions which specifically address the seismic strengthening of unreinforced masonry bearing wall buildings, and include a special procedure that is based on empirical evidence and is applicable to certain types of unreinforced masonry buildings. The 1991 UCBC has already been used to investigate, and retrofit when necessary, thousands of buildings in California, where the code is endorsed by local ordinances mandating the mitigation of seismic hazards from unreinforced masonry buildings. When lacking such endorsement, and concurrent legal protection, engineers have generally resorted to either conservative methods, the UCBC procedure, or other advanced analytical models, the final decision often being a matter of each engineer's beliefs in the seismic behaviour of unreinforced masonry buildings and their modelling. A review of the state of knowledge and current research on this topic is useful in this respect.

The objectives of this paper are (*i*) to present the theoretical background and practical applications of a new pro-

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cedure to evaluate unreinforced masonry bearing wall buildings, developed in California and recently integrated into the new CGSEEB; and (*ii*) to summarize the findings from other recent experimental and analytical research activities on the seismic behaviour of unreinforced masonry buildings, and, from advances in their modelling, to provide practising engineers and researchers with a clear overview of the state-of-the-art on this topic. The author's contribution herein is to report and comment on the information available, albeit scattered, in the published literature. A comprehensive presentation of concerns regarding the seismic performance of existing unreinforced masonry buildings in an eastern North American seismicity context, of the various issues that pertain to the seismic hazard of existing unreinforced masonry buildings, of the state-of-the-practice as required by various North American building codes and standards, of the identification of the known modes of failure of unreinforced masonry buildings, and of the recorded performance of unreinforced masonry buildings during recent earthquakes is available elsewhere (Bruneau 1994).

Existing knowledge on reinforced or partially reinforced masonry is not reviewed; current research on improved determination of various material properties (compressive, tensile, shear strengths of grout, mortar, brick and masonry, friction and bond strength, absorption rates, etc.) is not covered; studies of behaviour and performance not directly related to seismic response are not discussed. Findings exclusive to special types of structures (e.g., single-storey masonry houses or adobe houses) and construction (e.g., stone masonry, confined masonry) are not reported herein. The special problem of unreinforced masonry used as infill to reinforced concrete or steel frames is also beyond the scope of this study.

2. Historical overview

Prior to the Long Beach earthquake of 1933, unreinforced masonry construction in California was essentially identical to that found elsewhere across North America. The potential seismic vulnerability of unreinforced masonry buildings had been observed long before 1933 in California, but the widespread damage suffered by unreinforced masonry schools throughout this suburb of Los Angeles during the 1933 earthquake fuelled the necessary public outcry and support to pass California's Field Act which prohibited the use of masonry in all public buildings in the state. Before the end of 1933, the City of Los Angeles and many other cities had already outlawed all unreinforced masonry bearing wall construction. Gradually, unreinforced masonry became an archaic building construction type in California, and thereafter in the other states west of the Rockies. Research eventually emerged to develop, improve, legitimize, and promote structural systems built of reinforced masonry as a new, more ductile, and seismic-worthy construction, and, for a long time, little was done to improve the understanding of the seismic behaviour of unreinforced masonry construction. Still, California was left with a considerable inventory of unreinforced masonry buildings: Over 10 000 in the Los Angeles area, 4000 in the San Francisco area, and more than 10 000 in the rest of California (Seismic Safety Commission 1991). It is noteworthy that the active construction of unreinforced masonry buildings continued unhampered elsewhere throughout the United States and, until a few decades ago, in seismically active regions of Canada.

Eventually, some cities in California began to appreciate the magnitude of the threat to life safety posed by the existing seismically deficient infrastructure, and enacted various ordinances to address the earthquake hazards posed by unreinforced masonry buildings. Rapidly, the absence of reliable and accurate analytical models able to correlate with past damage observations, and the tremendous economic penalty ensuing from conservative safety assessments, pointed to the need for new research to develop effective seismic hazard mitigation models for unreinforced masonry buildings. One such mitigation methodology (ABK 1984) was developed in the early 1980s by a joint venture of three Los Angeles consulting engineering firms, Agbabian & Associates, S.B. Barnes & Associates, and Kariotis & Associates (ABK). It was developed further to a considerable amount of testing; although many nonlinear analyses have been conducted by ABK to validate results, the methodology is still considered by many as being largely empirical, i.e., fundamentally based on test results for given structural materials and building configurations. The advances in knowledge and innovations provided by this research effort were substantial, and were believed to provide more reliable seismic evaluations for unreinforced masonry buildings of a certain type commonly found in southern California. Understandably, the ABK methodology rapidly found acceptance there: it was adopted in 1987 as an alternate design method for seismic risk evaluation by the City of Los Angeles building code, and by the UCBC, a code specifically designed to address problems germane to existing structures. It was also endorsed by the ATC-14 (ATC 1987; Poland and Malley 1989), ATC-22 (ATC 1989*a*, 1989*b*), and the NEHRP Handbook for Seismic Evaluation of Existing Buildings (FEMA 1992*a*) with some minor modifications. These three documents propose a methodology for evaluating the seismic resistance of existing buildings in a broader scope encompassing all types of engineered constructions; the last two are in a format compatible with the NEHRP design recommendations (FEMA 1988). Finally, as the NEHRP document became a model for the CGSEEB, the ABK special procedure became integrated into the Canadian guidelines. Figure 1 provides a flow chart of this evolution. More detailed information on the code structure and seismic ordinance history of California is available elsewhere (Asakura 1987; Seismic Safety Commission 1990).

The concepts, research and models developed by the ABK group are subsequently reviewed, in the perspective of the CGSEEB. Comments on other research findings which confirm or expand on the ABK findings are added whenever appropriate. Findings on other aspects of the seismic behaviour of unreinforced masonry buildings needed for advanced studies beyond the scope of the CGSEEB, such as dynamic analysis and finite element analyses, are also reviewed in this paper.

3. Canadian guidelines for the seismic evaluation of existing buildings — special procedure for unreinforced masonry buildings

3.1. Purpose

Recent earthquakes have greatly contributed to raising awareness of the seismic hazards of unreinforced masonry buildings. The performance of such buildings in North America is extensively documented in published reconnaissance reports (Scholl and Stratta 1984; Shah et al. 1984;

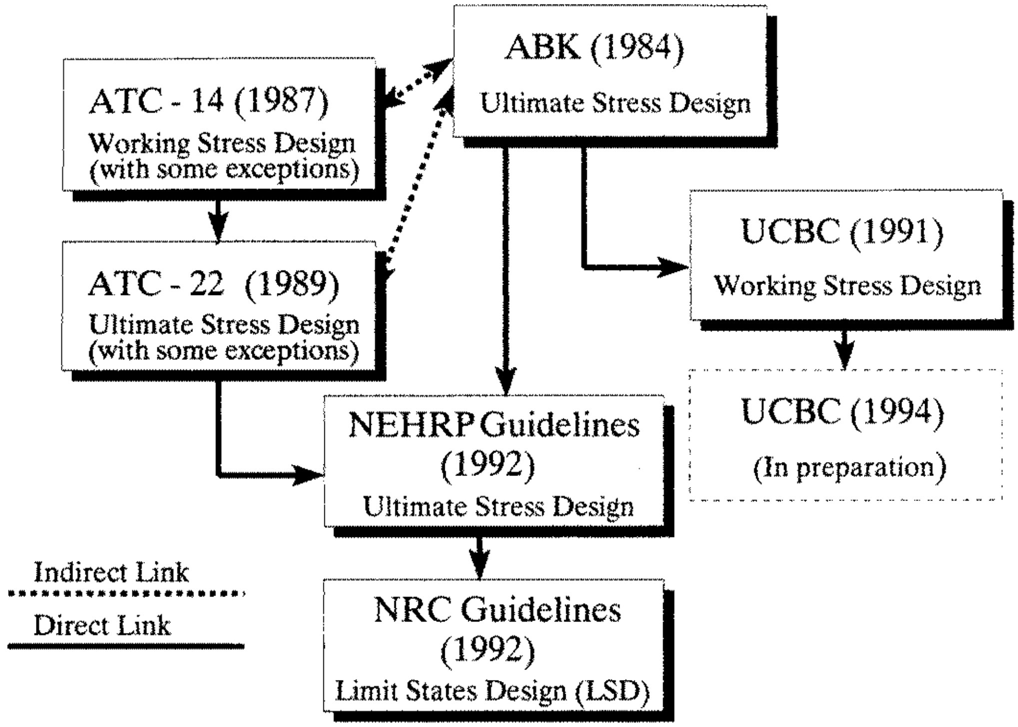

FIG. 1. Evolution of codified unreinforced masonry seismic evaluation procedure.

Reitherman et al. 1984; Kariotis 1984; Adham 1985*a*; Reitherman 1985; Swan et al. 1985; Esteva 1988; Hart et al. 1988; Deppe 1988; Moore et al. 1988; Muria-Vila and Meli 1989; Meli 1989; Mitchell et al. 1989; EERI 1990; Bruneau 1990; Cross and Jones 1991; Rutherford and Chekene 1991; Kariotis et al. 1991; EERI 1992, 1993; to name a few). Generally, with the exception of a comprehensive data collection by Rutherford and Chekene (1991) and a few other reports, damage surveys reported in the literature tend to concentrate on downtown cores, where the building stock consists mostly of older unreinforced masonry buildings, and may thus be biased toward higher damage (e.g., Shah et al. 1984). Still, in many instances, some unreinforced masonry buildings managed to survive earthquakes undamaged (Hart et al. 1988; Deppe 1988; Freeman 1932), often next to others that suffered extensive damage. There is obviously a relationship between the quality of construction and materials, peak ground accelerations, and damage; failures have been reported for peak ground acceleration as low as $0.1g$ in cases where the construction quality was very poor (Reitherman 1985). However, some large prestigious buildings of presumably higher quality construction have also been reported to suffer severe damage (Elsesser et al. 1991). As with other types of construction, the particular structural characteristics and layout of an unreinforced masonry building are responsible for its survival or failure.

An effective seismic evaluation and rehabilitation strategy must address all of the recognized common failure modes of unreinforced masonry buildings. These can be regrouped in the following categories:

- lack of anchorage;
- anchor failure;
- in-plane failures;
- out-of-plane failures;
- combined in-plane and out-of-plane effects; and
- diaphragm-related failures.

Of these, the potential out-of-plane failure of unreinforced masonry elements (parapet, veneers, gables, and unanchored walls) during earthquakes constitutes the most serious life-safety hazard for this type of construction. More details and examples on the particulars of each failure mode are available elsewhere (Bruneau 1994).

The current edition of the National Building Code of Canada (NBCC) (NRC 1990) prohibits the use of load bearing and lateral-load resisting unreinforced masonry in buildings located in regions where the peak ground acceleration or peak ground velocity may exceed $0.08g$ or 0.10 m/s respectively. This effectively bans new unreinforced masonry buildings from most Canadian cities. Moreover, although the NBCC is explicitly written for new buildings, many engineers consider compliance to the NBCC a minimum requirement when asked to evaluate the seismic resistance of existing buildings. On that basis, the above ban automatically makes noncompliant all buildings built prior to the enactment of the code. More importantly, it ensures that the NBCC will not provide any assistance to an engineer who needs to prevent the above failure modes. Furthermore, disregarding the above restrictions, attempts to achieve full compliance with the currently specified seismic force level of the NBCC for unreinforced masonry buildings can be equally frustrating, particularly since the CAN3-S304-M Masonry Design for Buildings (CSA 1984) recommends engineering analysis based on conservative but simple elementary principles of elastic mechanics of materials (coupled with some semi-empirical relationships to account for stability and load eccentricity effects). The United States standards and practices differ little from the above.

Indeed, past evaluations of the seismic-resistance adequacy of existing unreinforced masonry buildings have generally revealed their noncompliance with the Canadian design

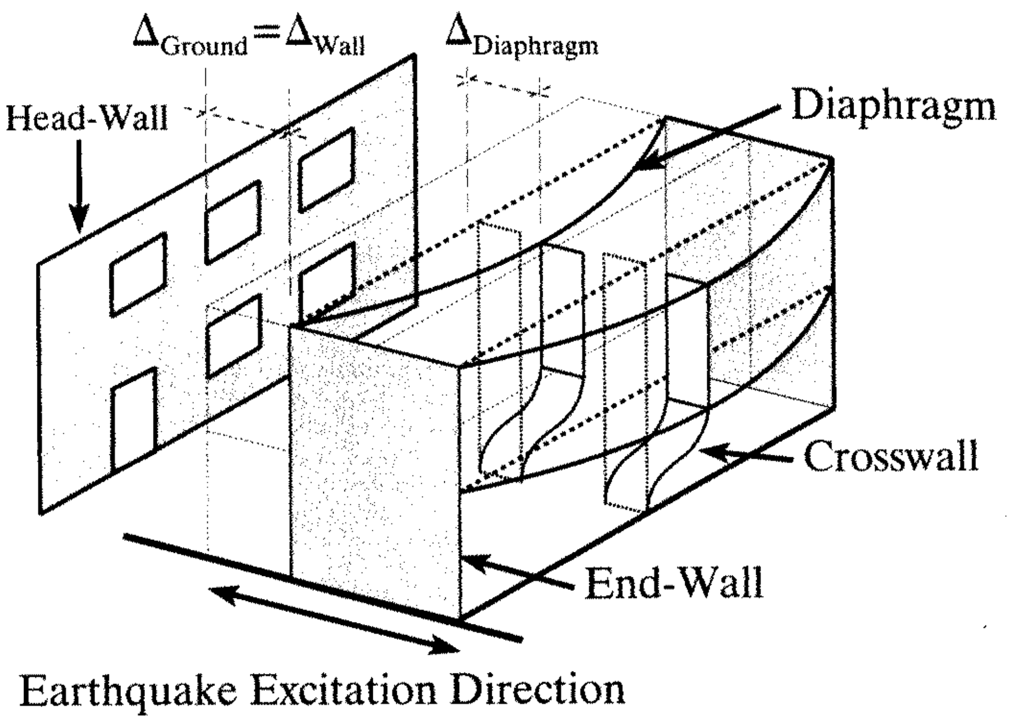

FIG. 2. Load path in unreinforced masonry building, and definitions.

codes and standards. In absence of alternative evaluation procedures, this verdict has often been misinterpreted as an indication that comprehensive seismic retrofit works must be urgently undertaken to remedy hazardous conditions. This approach fails to recognize the limitations in the applicability of modern codes, and their inappropriateness to deal with archaic materials and construction practices, and has led already, in some instances in eastern Canada, to the gutting of the entire interior unreinforced masonry structural systems of buildings and their reconstruction using contemporary structural systems and materials on which the "preserved" historical facade was then attached.

The CGSEEB is a document specifically designed to address problems germane to existing structures; it establishes acceptable life-safety requirements for buildings that undergo alterations or changes in use, by offering alternative methods of achieving safety so that the inventory of existing buildings can be preserved. The special procedure presented in Appendix A of the CGSEEB includes comprehensive seismic evaluation provisions for unreinforced masonry bearing wall buildings, but stops short of some U.S.A. documents, such as the 1991 UCBC, which also includes some strengthening provisions. The CGSEEB, UCBC, and other similar documents (Fig. 1) formulate a special procedure and methodology which standardizes the steps of a structural engineering investigation of the seismic-resistance adequacy of an existing unreinforced masonry building, and establishes strict guidelines against which this adequacy must be gauged. This methodology includes the following: recommended procedures for the acquisition of building information, including the reconstruction of engineering documents (should originals be missing) and the testing of components; a comprehensive review of the anchorage of wall elements; an evaluation of the dynamic stability of anchored unreinforced masonry wall elements; a lower design lateral-force level than that of comparable new buildings; a detailed review of the adequacy of diaphragms and unreinforced masonry walls; recommended capacities for structural elements and materials; and a progressive abatement of the breadth of the seismic mitigation effort for structures exposed to a lesser seismic risk.

Eliminating the possibility of having unreinforced masonry walls separate from the roof and floors during an earthquake, with the ensuing collapse of the structure or parts thereof, is the cornerstone of the procedure. Thus, should the absence or inadequacy of existing ties between unreinforced masonry walls and diaphragms be discovered during the structural evaluation, the methodology automatically calls for a retrofit requiring at least the installation of a new wall anchorage system, including the bracing of parapets. Without this structural integrity, the methodology is not applicable. It is noteworthy that, based on observations of past earthquake damage to unreinforced masonry buildings, this is a sound practice, relatively economical and easy to implement. It should be adopted as a minimum measure even by engineers upgrading unreinforced masonry buildings using other approaches. The special procedure of the CGSEEB is also limited to buildings that have flexible diaphragms at all levels above the base of the structure, a maximum of six storeys above the base of the building, and vertical elements of the lateral force-resisting system consisting predominantly of walls on at least two lines parallel to each axis of the building, although an open front on one side only is permissible for single-storey buildings. While the UCBC is finding broad acceptance in parts of California where unreinforced masonry buildings of this architecture are common, the objective of codes such as the UCBC and the CGSEEB is solely to mitigate the risk of life loss or injuries, even though any improvement in life safety is invariably accompanied by some reduction in property damage. Therefore, indiscriminate application of such a special procedure to heritage structures may not ensure a seismic performance meeting stringent preservation goals.

The CGSEEB methodology relies on some relatively new concepts describing the seismic behaviour of unreinforced masonry buildings. These are reviewed next.

3.2. Load path

The special procedure first assumes that the ground motion is directly transmitted unmodified to each floor by the end-walls parallel to the direction of earthquake excitation. This is equivalent to saying that these walls are infinitely rigid in-plane. Thus, each floor diaphragm is seismically excited at its end-attachment points to the unreinforced masonry walls by the original unamplified ground motion. These diaphragms, in turn, push on the head-walls (i.e., the walls perpendicular to the ground motion direction, also sometimes referred to as normal walls) which are excited in their out-of-plane direction. Therefore, the dynamic characteristic of the diaphragms directly influences the severity of the out-of-plane excitation of the head-walls and the required strength of wall-to-diaphragm anchors. Figure 2 illustrates this load path and its terminology.

The methodology imposes limits on diaphragm spans (as expressed by demand-to-capacity ratios) to control the severity of the diaphragm-amplified seismic excitations imparted to the unreinforced masonry head-walls. Similarly, limits on slenderness ratios derived from dynamic stability concepts also aim at protecting these head-walls against out-of-plane failure. Some of these concepts significantly depart from conventional structural engineering thinking, and can be controversial.

3.3. Unamplified earthquake excitation of end-walls

This is by far the most controversial aspect of the special methodology. It is indeed difficult to accept that unreinforced masonry walls could be evaluated using an equivalent static seismic force level lower than that used for the design of new nominally ductile steel and concrete structures. A few different explanations are found in the existing literature to substantiate this concept. More specifically, they are the following:

(*i*) Foundation uplift effects can attenuate seismic response down to nearly the input acceleration level (Yim and Chopra 1983; SEAOC 1986; Nakaki and Hart 1992). In the original ABK study (ABK 1984), various walls were modelled as rigid elements supported on bilinear nonlinear foundation springs capable of carrying only compression loads, and taking impact damping into account. For various wall aspect ratios (of height-to-width ratio up to 1.5) and soil stiffnesses, displacement amplifications of less than 10% were recorded. As flexible walls typically amplify ground motions less than rigid walls, it was concluded that overall amplification of ground motion along the height of unreinforced masonry walls could be neglected (ABK 1984; SEAOC 1991).

(*ii*) The pseudo-acceleration response spectrum of a single-degree-of-freedom oscillator reduces to the peak ground acceleration value for very stiff structures, such as low-rise unreinforced masonry buildings with substantial walls, a situation that is conservatively neglected in design codes. However, this particular reasoning has been challenged by recent evidence gathered from an instrumented unreinforced masonry building which survived the 1990 Loma Prieta earthquake (Tena-Colunga 1992) where very large amplifications of the ground base accelerations were recorded at the floor levels of a two-storey unreinforced masonry building.

(*iii*) The intent of the methodology is the mitigation of damage, not its elimination. Hence, a larger risk can be accepted in light of the difficulties and high costs (per square foot, but also as a percentage of assessed value) of retrofitting ordinary unreinforced masonry buildings (Asakura 1987; Zsutty 1991).

It is noteworthy that for rare and severe earthquake excitations, the force in the walls cannot exceed that transmitted by the diaphragms yielding in shear, in addition of course to the walls' own inertia contribution. In that case, the governing in-plane loading condition for end-walls is conservatively obtained by considering simultaneous yielding of the diaphragms at all storeys (SEAOC 1991; Zsutty 1991).

In addition, the CGSEEB recognizes that masonry structures on soft soils have suffered more damage in the past (e.g., Bruneau 1990; Housner 1990) and that important structures should be designed to a lower anticipated damage level, by introducing an effective velocity ratio, v', defined such that

$$[1] \qquad v' = \frac{vIF}{1.3} \leq 0.4I$$

where v is the zonal velocity ratio, I the importance factor, and F the foundation factor typically found in the NBCC. The upper bound limits the design force level to the maximum used in California, multiplied by the importance factor. The following discussion on diaphragms is necessary before the wall's design force requirements can be presented and understood.

3.4. Diaphragm response

As part of the ABK research program, static and dynamic tests of floor diaphragms (ABK 1981*a*) and unreinforced masonry walls for out-of-plane motion (ABK 1981*b*) were conducted, with a particular emphasis on the acquisition of data on their nonlinear behaviour and energy dissipation (ductility) characteristics. These components were selected in light of the limited or absent knowledge of their ultimate behaviour, and their considerable impact on the overall seismic resistance of unreinforced masonry buildings. Flexible diaphragms' behaviour was of particular interest as the out-of-plane excitation of walls in unreinforced masonry buildings proceeds through the floor and roof diaphragms.

Fourteen different diaphragms were tested to obtain information on their in-plane static and dynamic, linear and nonlinear, properties. Eleven diaphragms tested were of wood construction, the others being of steel decking either filled with concrete or not (Adham et al. 1978; ABK 1981*a*). Strong diaphragms in the elastic range are known to respond essentially as 2% damped oscillators, producing large amplifications of up to 3 or 4 times the input accelerations, velocities, and displacements (SEAOC 1986; Kariotis 1989). However, flexible diaphragms were found to have a highly nonlinear hysteretic behaviour if excited at their edges past a given threshold of effective peak acceleration, which in some cases could be as low as $0.1g$. This nonlinear behaviour has a most positive effect in reducing the diaphragm's peak accelerations and velocities at mid-span and, in turn, the out-of-plane excitation of the unreinforced masonry walls. Typically, it was found that for buildings with crosswalls, and buildings without crosswalls but within a certain range of diaphragms' strength and spans properties, for input earthquake excitations scaled to $0.4g$, the possible amplification of the input seismic velocity at the diaphragm's midspan

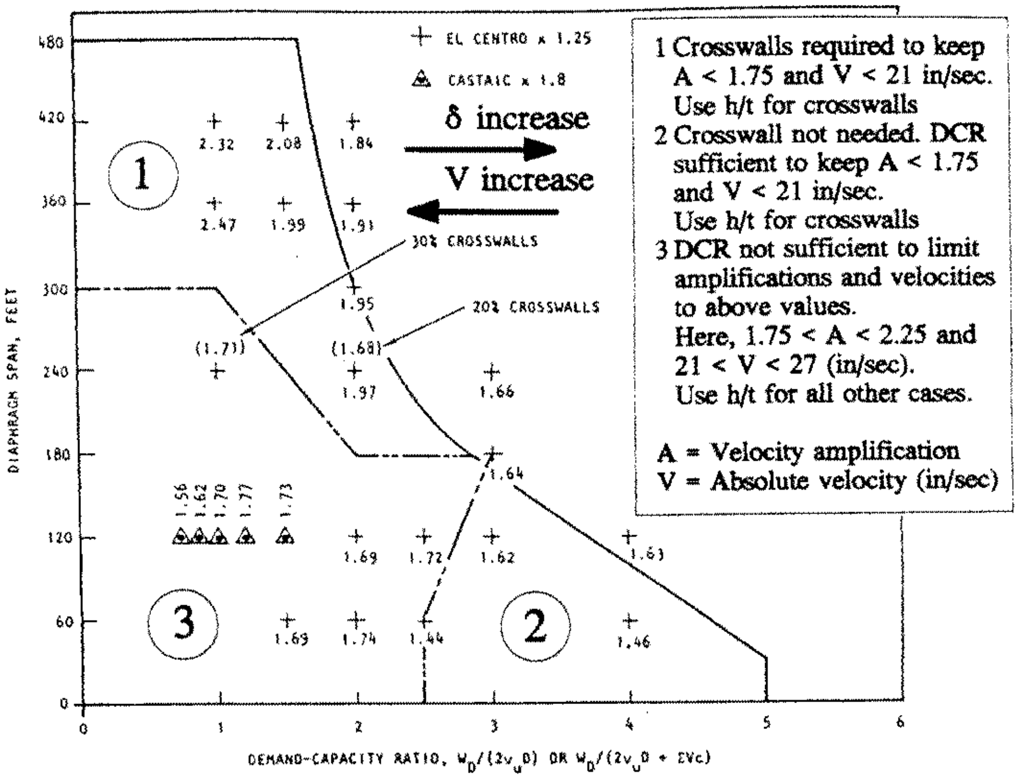

FIG. 3. Experimentally obtained velocities superposed on figure of acceptable diaphragm span versus demand–capacity ratio (adapted from ABK 1984).

drops to 1.75. For all other types of flexible diaphragms considered under the ABK test program, typical amplifications reduce to 2.0 and 2.25 for roof and floor constructions respectively. Some experimentally obtained amplification factors are overlaid on the acceptable diaphragm span figure of the CGSEEB (Fig. 3). Some measured peak relative diaphragm displacements are presented in a similar format in Fig. 4. It is noteworthy that the extreme boundary of that figure corresponds to a diaphragm centre span displacement of 125 mm (5 in.). Numbers in parentheses have been obtained by nonlinear structural analyses using phenomenological models of diaphragms and an assumed crosswall nonlinear model, to illustrate the effectiveness of crosswalls in attenuating seismic velocity amplifications (ABK 1984; SEAOC 1986).

Crosswalls are, exclusively, the wood partitions often present in most existing unreinforced masonry buildings. Although their original purpose is nonstructural, if properly connected at their ends to the floors above and below, as they often are, they can act as excellent energy dissipators capable of attenuating the diaphragm's seismic velocity amplification. To achieve this result, crosswalls are recommended to have a minimum yield strength of at least 30% of the diaphragm's yield capacity (i.e., to ensure a minimum adequate hysteretic energy capability) and to be spaced at no more than 12.5 m (40 feet) from each other in the direction of the span. This advantage of crosswalls is, however, limited to long-span diaphragms, as delimited by region 1 of Figs. 3 and 4.

It is significant that no data exist on the seismic behaviour of diaphragms excited beyond the range encompassed by Figs. 3 and 4. In severe earthquakes, it is essential to ensure that the existing structure falls within these boundary limits to avoid the risk of excessive inter-storey displacements and velocity amplifications. The demand–capacity ratio, DCR, is a normalized parameter that relates the inertia forces produced by a $1.0g$ dynamic response of the diaphragm to its yield level (note that the flexible diaphragms considered are all yielding in shear). For a diaphragm without crosswalls, DCR can be calculated as

$$[2] \qquad \mathrm{DCR} = \frac{2.5v'W_{\mathrm{d}}}{\sum v_{\mathrm{u}}D}$$

This ratio is not unlike other seismic-intensity-to-yield-strength normalization factors commonly used in seismic response studies of nonlinear inelastic structures (Mahin and Lin 1983). The previously reported seismic input amplifications of up to 2.25 obtained experimentally in diaphragms have been interpreted by some (Zsutty 1991) as being indicative that an amplified elastic diaphragm acceleration response of $0.9g$ is expected for an earthquake excitation of $0.4g$, even though the ABK methodology amplification levels pertain solely to seismic velocities. In the CGSEEB context, the ambiguity disappears, as the seismic acceleration and velocity zones are mostly proportionally divided. For example, velocity zone 6 is assigned a velocity ratio of 0.4 m/s, whereas a peak ground acceleration of $0.4g$ can be logically expected in acceleration zone 6. Since the demand–capacity ratio is an arbitrary normalization parameter, the demand side of the equation was rounded up to $1.0g$ in the ABK methodology, or, equivalently, $2.5v'$ in the CGSEEB. Note, however, that the elastic diaphragm response value is used mainly for convenience, and that the maximum inelastic acceleration response of this diaphragm was coincidentally found to be $0.4g$ for an input acceleration of $0.4g$ (ABK 1984; SEAOC 1991).

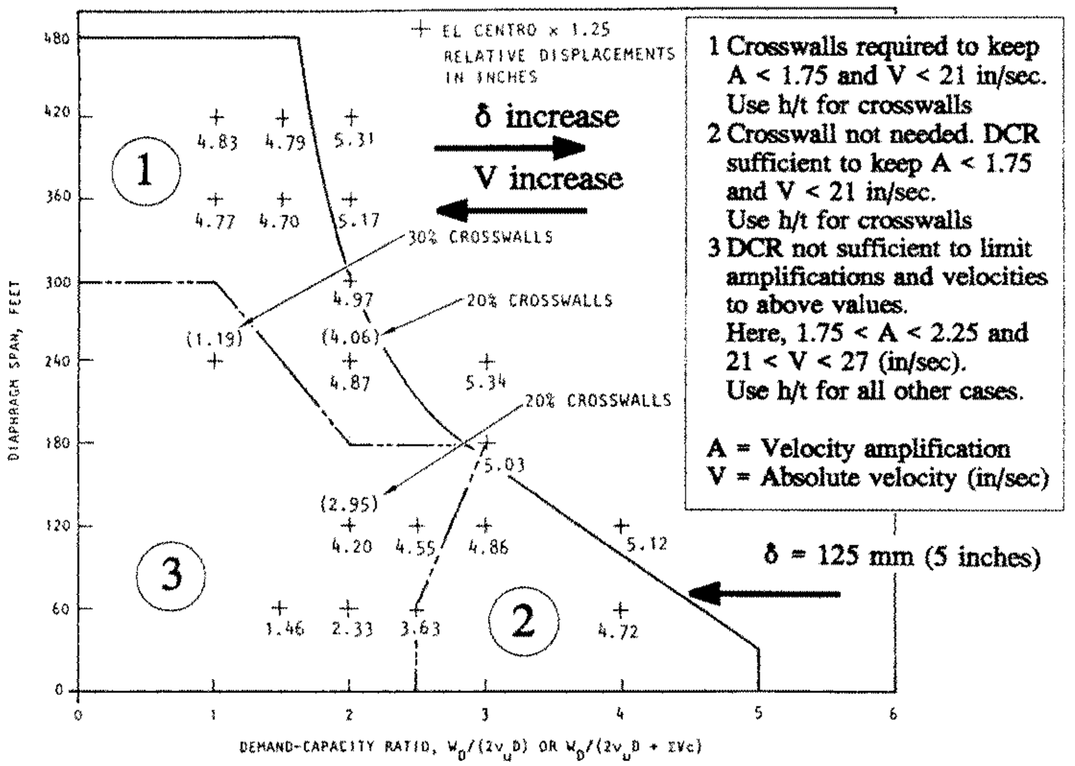

FIG. 4. Experimentally obtained displacements superposed on figure of acceptable diaphragm span versus demand–capacity ratio (adapted from ABK 1984).

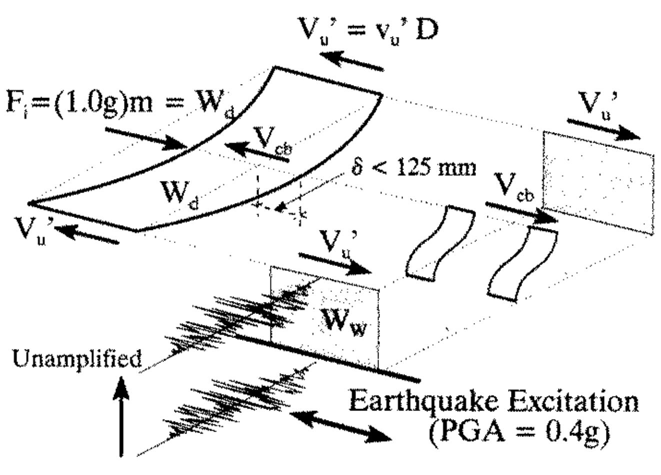

FIG. 5. Free-body diagram of yielded diaphragm and crosswall system.

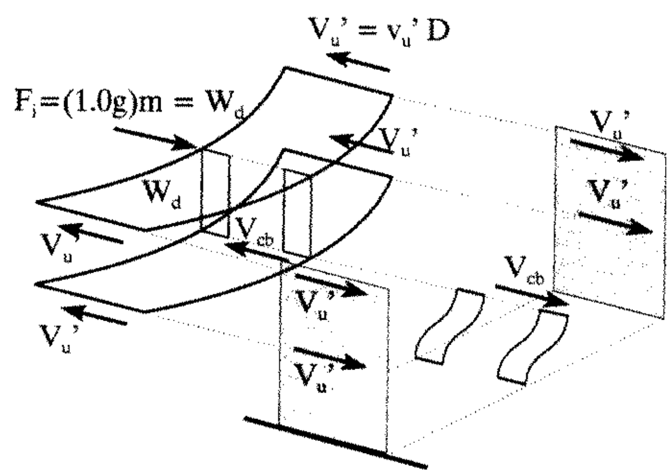

FIG. 6. Free-body diagram of yielded super-diaphragm and crosswall system.

As an alternative way to understand the demand–capacity ratio concept, Zsutty (1991) expressed it as a ductility ratio such that

$$[3] \qquad \mathrm{DCR} = \frac{\delta'}{\delta_u'} = \frac{F_{\delta'}}{F_u'} = \frac{m_d(1.0g)}{2V_u'} = \frac{1.0W_d}{\sum v_u D} = \frac{2.5v'W_d}{\sum v_u D}$$

where the ratio of the maximum elastic displacement, δ', to the yield displacement, δ_u', can be related to the elastic response force level, $F_{\delta'}$, over the yield force level, F_u', much like what is done conceptually to derive seismic force reduction factors. From this, the use of free-body diagrams becomes possible, as shown in Fig. 5. Isolating the yielding diaphragm subjected to an inertia force of $1.0g$ (demand), the reacting capacities can be easily summed up to produce the above equation.

When crosswalls are present, the demand–capacity ratio, DCR, can be similarly calculated as

$$[4] \qquad \mathrm{DCR} = \frac{2.5v'W_d}{\sum v_u D + V_{cb}}$$

where V_{cb} is the total shear capacity of crosswalls in the direction of analysis immediately below the diaphragm level being investigated, as shown in Fig. 5. For diaphragms in a multistorey building with qualifying crosswalls in all levels, a super-diaphragm concept can be formulated by assuming all diaphragms above the level under consideration to be

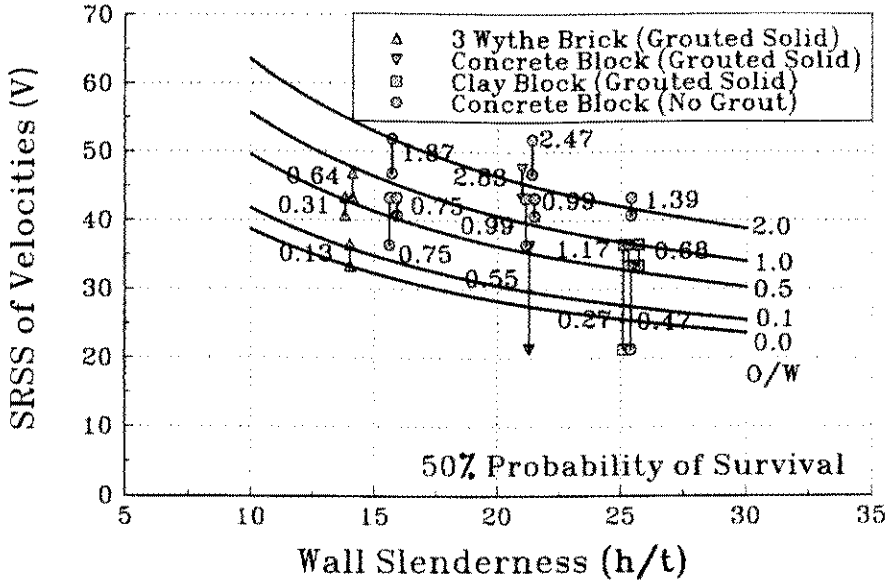

FIG. 7. ABK's test results for out-of-plane dynamic stability response of unreinforced masonry walls and regression analysis curves for 50% probability of survival. (The numbers are overburden to wall weight ratios; SRSS denotes the square root of the sum of the squares.)

yielded (i.e., $\Sigma\Sigma v_u D$), along with the crosswalls in a single-storey, as schematically illustrated in Fig. 6. This concept reflects that qualifying crosswalls, although weak, are usually much stiffer than the diaphragms and will not deform significantly when spanning between two diaphragms. Obviously, in that case, the demand–capacity ratio, DCR, is

$$[5] \quad \mathrm{DCR} = \frac{2.5v'\sum W_d}{\sum\sum v_u D + V_{cb}}$$

The demand–capacity ratio must obviously be calculated at each level (including the roof) for the set of diaphragms at and above the level under consideration. Other than DCR calculations, conventional diaphragm analysis is not required by the CGSEEB.

It is noteworthy that failure of the flexible diaphragm itself is rarely observed following earthquakes. This could be attributed in part to the tendency of earthquake reconnaissance teams to report observations made mostly from the exterior of buildings. In most cases, damage to the diaphragm itself would not impair its gravity load carrying capacity. However, since very flexible floor diaphragms could also behave as deep beams spanning between unreinforced masonry walls, the in-plane rotation of the diaphragm's ends can induce damage at the walls' corners. This form of damage is neglected by the special procedure, since it has no impact on life safety. More importantly, though, the absence of a good shear transfer between diaphragms and reaction walls can also account for damage at the corners of walls, especially in long narrow buildings for which the diaphragm in-plane shear forces cannot be transmitted over the small length of wall; the diaphragm will instead find its support by pushing on the unreinforced masonry walls in the transverse direction, a dangerous condition.

Finally, an empirical equation has been proposed to calculate the equivalent diaphragm length and the demand–capacity ratio of single-storey open front buildings. However, the author has not been successful in finding convincing analytical or experimental evidence in the existing literature to validate this concept. This concept is presented elsewhere (NRC 1992; ABK 1984; SEAOC 1986).

3.5. Out-of-plane wall response — dynamic stability

3.5.1. Concept

Joist-to-wall anchors provide out-of-plane support to the walls. If present in sufficient numbers and strength, these anchors will transform the out-of-plane behaviour of the unreinforced masonry walls, from tall unrestrained cantilevers to shorter one-storey high panels dynamically excited at each end by the floor diaphragms. Unreinforced masonry buildings are most vulnerable to flexural out-of-plane failure. Furthermore, whereas an in-plane failure often does not endanger the gravity load carrying capabilities of a wall, an unstable and explosive out-of-plane failure could.

Parapet failures fall into this category. These nonstructural unreinforced masonry elements behave, if unrestrained, as cantilever walls extending beyond the roof line. Being located at the top of buildings, they are subjected to the greatest amplification of the ground motions, and are consequently prone to flexural failures. Gables of churches and other buildings, when improperly anchored to the roof, behave much like parapets. Multi-wythe walls improperly bonded along their collar joint (e.g., no or discontinuous mortar) are also extremely vulnerable, each wythe acting independently as an individual thin wall. The exterior layers without contact to any other structural components will usually fail first, at a very low level of seismic excitation. This is also true for unreinforced masonry veneers.

The concept of dynamic stability is relatively new. It was developed following extensive testing and analytical work and was first introduced in the ABK methodology to assess the out-of-plane seismic resistance of unreinforced masonry walls (ABK 1981*a*, 1981*b*, 1984; Kariotis et al. 1985; Adham 1985*b*). It was formulated following observations that unreinforced masonry walls properly anchored to floors and roof diaphragms can resist more severe earthquakes than otherwise predicted by traditional static analysis methods. After cracking, portions of a wall behave as rigid-body members rocking on the wall's through-cracks; if gravity forces are sufficient

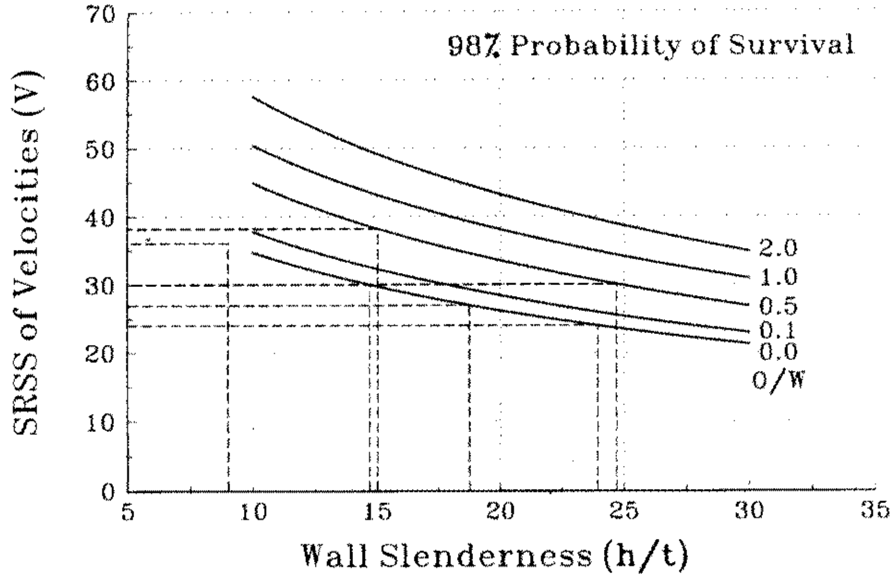

FIG. 8. ABK's regression analysis curves for 98% probability of survival for out-of-plane dynamic stability response of unreinforced masonry walls (calculation of permissible limits: SRSS denotes the square root of the sum of the squares).

TABLE 1. Limit of height-to-thickness ratios as per different sources, in their most severe seismic zone (i.e., typically in regions mapped at 0.4g peak ground acceleration for a 10% probability of exceedence in 50 years)

	Buildings of DCR* and spans as specified for limited velocity amplification of 1.75†			All other buildings		
Condition	Needed	ABK	UCBC ATC-22 NEHRP CGSEEB	Needed	ABK	UCBC ATC-22 NEHRP CGSEEB
Walls of single-storey buildings	22	20	16	18	14	13
Multistory buildings						
First storey walls	>30	20	16	25	20	15
Top storey walls	14	14	14	9	9	9
All other walls	25	20	16	15	15	13
Parapets	—	1.5	1.5	—	1.5	1.5

*DCR denotes demand–capacity ratio.
†That is, in region 1 of DCR figure if crosswalls are present, or region 2.

to prevent overturning of these individual bodies through the entire earthquake, a condition of *dynamic stability* exists.

3.5.2. Experimental validation

Three walls of three-wythe common brick, five of clay block, and twelve of concrete blocks, of height-to-thickness ratios ranging from 14 to 25, were tested under dynamic out-of-plane excitation of progressively increasing intensity, for a total of 194 tests. Actuators, located at the top and bottom of the walls to be tested, imparted them dynamic displacements time-histories. Nonlinear analyses of buildings were conducted, assuming rigid in-plane response of unreinforced masonry elements and taking advantage of the nonlinear hysteretic properties of flexible diaphragms, to derive sets of dynamic displacement time-histories (Asakura 1987). The magnitude of the compressive gravity load (overburden) simultaneously applied was also varied. Selected data points extracted from the ABK results (ABK 1981*b*) are presented in Fig. 7. The lower and higher points in each data set indicate the last dynamically stable and first dynamically unstable points obtained as earthquake intensity was increased. Numbers next to each set represent the magnitude of overburden weight to self-weight of the walls tested. From these tests, increases in peak seismic input velocities at the base and the top of the walls were found to be detrimental to their resistance against out-of-plane collapse, whereas larger compressive loads were favourable. Input velocities bear a direct relationship to the seismic energy imparted to the system. However, for convenience, to systematically account for both the wall's top and bottom contributions to velocity excitation, the square root of the sum of the squares of these velocities is selected as the descriptive parameter. Finally, wall slenderness was found to be significant, but type of construction was not.

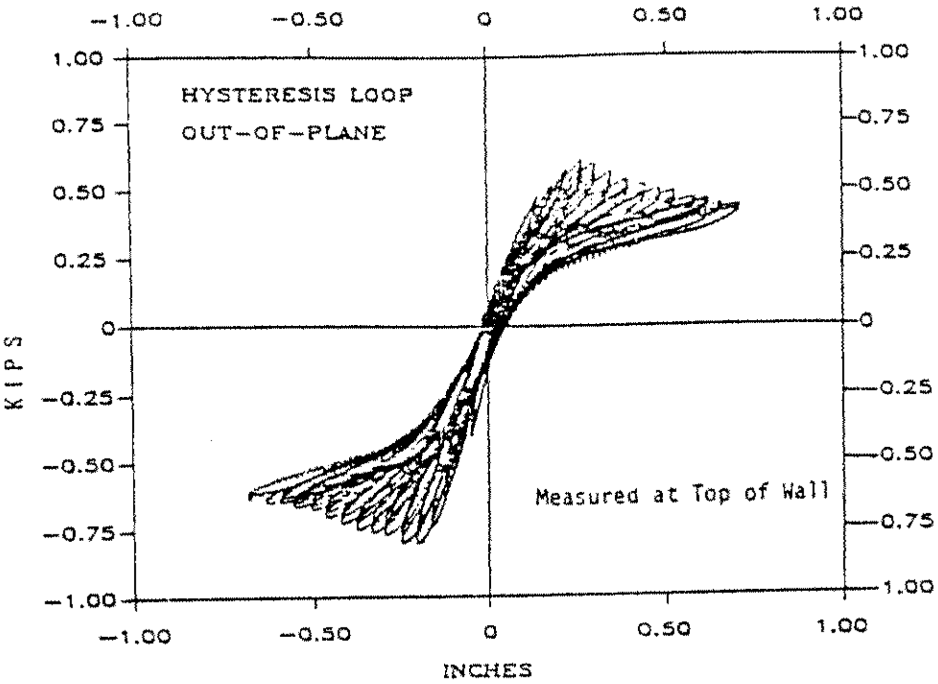

FIG. 9. Typical hysteretic curve of applied out-of-plane shear force versus top wall displacement for rocking wall (Prawel and Lee 1990*b*).

From these data, nonlinear regression analysis was conducted. The results for a 50% probability of survival are plotted on top of the data in Fig. 7, and for a 98% probability of survival in Fig. 8. The latter curves were used to develop the slenderness limits provided in the special methodology (Table 1), assuming the following: a peak ground velocity of 0.3 m/s (12 in./s), the ABK diaphragm velocity amplification factors previously reported, and an overburden (O/W) of 0 at any top storey and 0.5 elsewhere. In the ABK methodology, an upper bound value of 20 was adopted, even when higher values could be justified by dynamic stability concepts, to respect the limits specified for the empirical design of new masonry buildings (e.g., ICBO 1991*b*; ACI–ASCE 1988). With few exceptions, these limits have been further tightened to various degrees when adopted by the UCBC and subsequent methodologies.

It should be clear that the above procedure is applicable to walls able to *dynamically stably* rock as a single unit. This explains why the CGSEEB requires that all wythes be adequately interlocked by header bricks and collar joint. Otherwise, *h/t* ratios of individual wythe should be used (if properly anchored to each diaphragm).

Other researchers have experimentally corroborated the validity of the dynamic stability concept. In one study, Prawel and Lee (1990*b*) applied both static cyclic shear and dynamic shake-table excitation in the out-of-plane direction of unreinforced brick masonry walls under heavy compressive loading; these walls were fixed and pinned at their base and top respectively. They reported that (*i*) following the onset of the first horizontal full-width crack, additional loading was required to propagate that crack through the thickness and initiate a rocking rigid-body motion about the horizontal cracks; (*ii*) in some cases, 25% reserve capacity existed beyond first cracking, with lateral displacements reaching twice the cracking value at that capacity; and (*iii*) although the material itself is brittle, rocking produced hysteretic behaviour with recorded maximum lateral displacements up

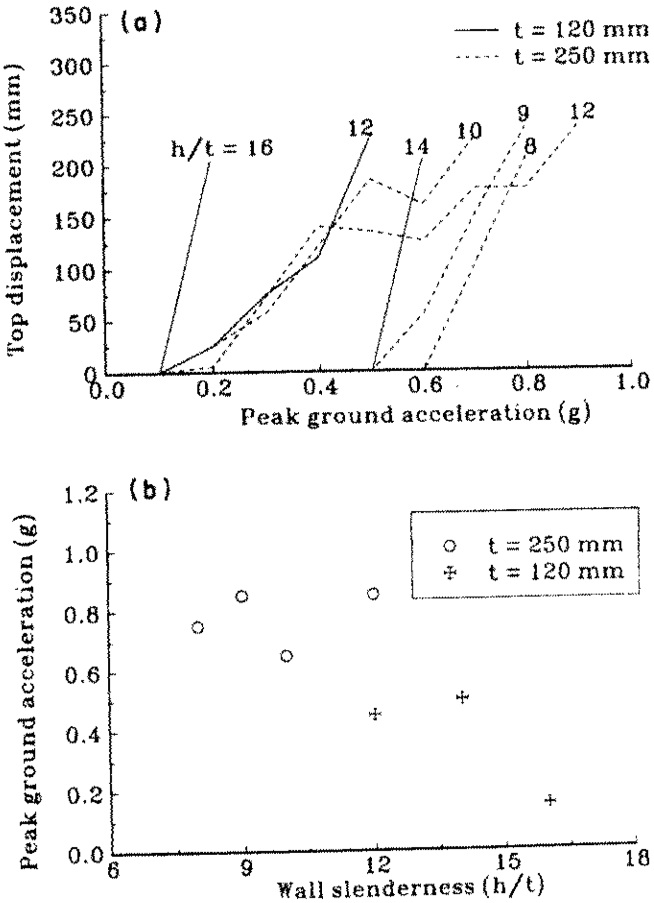

FIG. 10. (*a*) Out-of-plane wall top displacement versus peak ground acceleration of the scaled Lima 1970 N82W earthquake; (*b*) peak ground acceleration of that same earthquake at out-of-plane failure, as a function of wall slenderness (adapted from Bariola et al. 1990).

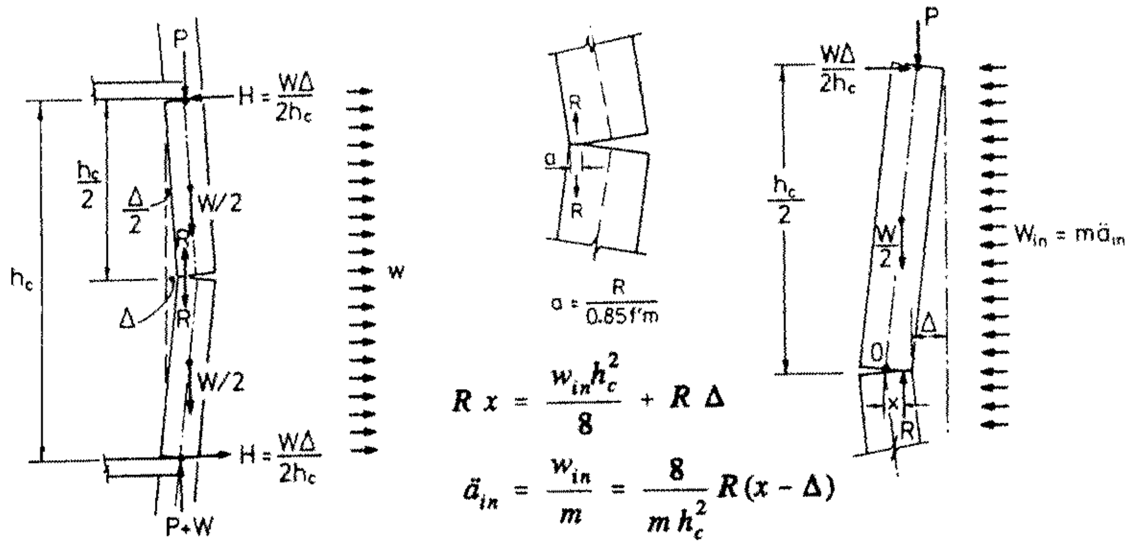

FIG. 11. Out-of-plane dynamic stability concept and equations, including location of applied forces and reactions, contact surface at mid-height, and equilibrium of forces on rigid-body wall segment (adapted from Priestley 1985).

to five times the cracking displacement. Figure 9 attests to this energy dissipation.

Bariola et al. (1990) also performed shake-table tests of unreinforced clay–brick walls; the walls were free-standing cantilevers of variable thickness (120 and 250 mm) and slenderness (8 to 16). Seven specimens were excited in their out-of-plane direction by the Lima 1970 N82W earthquake record, scaled to progressively higher intensities. With increasing earthquake severity, the seismic response of the walls evolved from a purely elastic cantilever response to a rigid-body rocking mode subsequently to cracking at their base; ultimately, walls collapsed by overturning. The out-of-plane rocking of the tested walls was considerable and sometimes allowed them to survive earthquakes more than four times more intense than those that caused cracking, as illustrated in Fig. 10*a* (in that figure, the elastic displacements are imperceptible in comparison with the large inelastic rocking displacements). No precise correlation could be seen between the peak ground accelerations that produced failure and the slenderness of the walls. Finally, for a given height-to-thickness slenderness ratio (*h/t*), thicker walls can apparently survive more severe earthquakes, not only from a strength of mechanics uncracked point of view, but also in their post-cracking range as their rocking behaviour seems more stable (Fig. 10*b*). These experimental results add credibility to the dynamic stability concepts.

3.5.3. Supplementary analytical approach

Priestley (1985) proposed an alternative hand-calculation method based on energy equivalence to predict the dynamic out-of-plane wall stability. Assuming in-phase floor accelerations and a condition of average constant acceleration acting on walls conservatively taken as spanning in a simply supported fashion between storeys, simple relationships of static equilibrium were developed on the rigid-body wall segment post-cracking, as shown in Fig. 11. Essentially, in this method, the location of the gravity forces (overburden, *P*, and wall self-weight, *W*) along the wall, and the reaction forces needed for stability, must be located and calculated, the width of the bearing surface at mid-span must be determined from equilibrium conditions, the displacement conditions for instability must be established from geometry, and finally, the formulation of equilibrium equations allows the expression of the inertia forces as a function of all other variables. The key equations resulting from all these steps are presented in Fig. 11.

The relationship between the wall's acceleration (i.e., inertia force) and its displacement at mid-height is described as nonlinear elastic; it draws a linear curve until first cracking of the wall, reaches a maximum acceleration corresponding to a point of maximum static stability, and progressively returns to zero under much larger displacements. Since the area under this curve is associated with the total energy needed to fail the wall, Priestley suggested that a linear elastic model, whose ultimate limit would be selected to yield the same energy to failure as the actual nonlinear model, would be a good indicator of dynamic stability. It is thus assumed that if no single pulse of excitation requires an energy exceeding that required statically to produce collapse, then stability is ensured.

In a comprehensive illustrative numerical example of this method as applied to a five-storey unreinforced masonry building, Priestley found that high values of floor accelerations were needed to produce failure, provided that good quality masonry and positive anchorages of walls to floors and roof were present. The possible amplifications of the ground acceleration along the height of the building, as normally done with standard buildings, were considered in that example, contrary to the CGSEEB recommended special procedure. Also, while the ABK research dismissed the possible impact of vertical accelerations on the seismic performance of unreinforced masonry buildings, because of their frequency content particularities (Kariotis et al. 1985), Priestley elected to reduce gravity loads by an arbitrary allowance to account for these vertical accelerations. Assuming that response is dominantly driven by the first fundamental vibration frequency, Priestley found that lower overburden and less thickness at the top storey of tall bearing-wall structures, where the largest floor accelerations also occur when amplification is assumed possible, make the out-of-plane failure of unreinforced masonry walls more probable at that top level, as frequently observed following earthquakes. This appealing equivalent-energy method remains to be verified experimentally.

3.6. In-plane seismic loading of unreinforced masonry walls

Once it is understood that unamplified ground motion is applied at the diaphragm's edges, the in-plane seismic loading condition acting on walls simply becomes the inertia force produced by the tributary mass times the base ground acceleration, or simply the zonal acceleration factor (or zonal velocity in the Canadian system) times the tributary weight. Moreover, the reactive contribution of diaphragms obviously cannot exceed that produced by diaphragm shear yielding.

Consequently, for buildings without crosswalls, the wall storey force distributed to a shear wall at any diaphragm level, F_{wx}, is

$$[6] \quad F_{wx} = v'(W_{wx} + W_d/2)$$

but need not exceed

$$[7] \quad F_{wx} = v'W_{wx} + v_u D$$

For buildings with crosswalls at all levels, the value of F_{wx} is 75% of that calculated by 6, to account for the damping and drag effects of the crosswalls, but need not exceed

$$[8] \quad F_{wx} = 0.75v'\left[W_{wx} + \sum W_d\left(\frac{v_u D}{\sum\sum v_u D}\right)\right]$$

or

$$[9] \quad F_{wx} = 0.75v'W_{wx} + v_u D$$

where W_d is the total dead load tributary to a diaphragm, including walls perpendicular to the direction of motion; ΣW_d is the total dead load tributary to all of the diaphragms at and above the level under consideration; W_{wx} is the dead load of unreinforced masonry wall assigned to level x halfway above and below the level under consideration; v_u and D are respectively the unit shear strength and depth of the flexible diaphragm; and $\Sigma\Sigma v_u D$ is the sum of shear capacities of both ends of diaphragms coupled at and above the level under consideration for diaphragms coupled with crosswalls. These equations are self-explanatory in light of Figs. 5 and 6. Obviously, the wall storey shear is equal to the sum of the wall storey forces at and above the level under consideration, as in usual codified seismic analysis.

3.7. In-plane seismic resistance of unreinforced masonry walls

Excessive bending or shear may produce in-plane failures, depending on the aspect ratios of the unreinforced masonry elements. For unreinforced masonry walls, shear in-plane failures are commonly expressed by double-diagonal (X) shear cracking. Fortunately, until the shear cracks become unduly severe, the gravity load carrying capacity of the walls is not jeopardized. In masonry facades having numerous window openings, spandrels and the short piers between those spandrels may also fail in shear. Usually the failure of one modifies the structural behaviour significantly enough to preclude that of the other.

Flexural failure of these structural elements is also possible, particularly for slender unreinforced masonry columns; the resulting cracking at both ends of an unreinforced masonry element transforms it into a rigid body of no further lateral load resisting capacity, unless gravity forces can provide a stabilizing effect. In light of this behaviour, a number of analysis strategies appear possible, as explained below.

3.7.1. Piers-only models

First, a legitimate, yet conservative, model is to assume that the spandrel beams in a perforated wall will crack under a very low lateral load, leaving the piers alone to resist the lateral loads (solid-pier/cracked-spandrel model). In this case, unreinforced masonry spandrels either behave as beams or deep-beams, failure initiating when the flexural stresses exceed the tensile strength of the brick at one course from the beam's bottom surface (the cracking of the lowest course being governed by the weaker tensile strength of the mortar) or when the shear stresses at the head joint near the beam's end exceed the mortar's bond strength, whichever comes first, as demonstrated by Abu-El-Magd and MacLeod (1980); in both cases, the failures are brittle. The behaviour of unreinforced masonry spandrels is thus completely characterized by the tensile and shear bond strengths, the tensile strength of bricks, and the relative depth (i.e., geometry) of the spandrel.

This approach is not unlike that recommended by some researchers for the analysis of reinforced masonry walls having numerous openings, where the masonry above and below the openings is neglected (Englekirk and Hart 1984). Although this models the structure at its ultimate state if the spandrels are shallow or not well connected to the piers, it immediately assumes a structure in its degraded condition, neglecting the potentially larger capacity of the structure before cracking. Analytical seismic-performance assessments of existing structures known to have survived major earthquakes have proven this approach to be excessively conservative (Boussabah 1993).

3.7.2. Wall models

Walls, or piers if the spandrels of perforated walls are deep and (or) of short span, may fail in flexural cracking, shear cracking (diagonal tension), or compression crushing. Traditionally, design standards recommend a no-tension criterion for the design of unreinforced masonry walls in flexure, or alternatively, in the absence of significant axial compression, the design is governed by the allowable tensile stresses, f_t. Resistance to shear stresses is checked independently, against an allowable shear stress enhanced by the presence of axial compression, if present (Bruneau 1994). Thus, in those cases, elementary equations of mechanics of materials are used, as illustrated in Fig. 12*a*. Fortunately, a number of other models of this behaviour are suggested in the existing literature.

3.7.2.1. Shear failure

Many researchers (e.g., Mayes and Clough 1975; Turnsek et al. 1978; Zingali 1986) have used a shear failure criterion directly related to the diagonal tension capacity by principal stresses relationships. Directly from Mohr's circle, for an unreinforced masonry panel in pure shear by diagonal tension, the normal tensile stress capacity, σ_t, of masonry is equal to the maximum shear stress at failure, τ_{max}, where $\tau_{max} = 1.5\tau_{mean}$ for a rectangular cross section and τ_{mean} is the maximum average shear stress (Fig. 12*a*).

The ultimate shear capacity of this same unreinforced masonry panel, stressed in combined shear and axial compression by earthquake excitation and gravity loading respectively, is also amenable to a single expression. Using the well-known classical expression for principal tensile stresses

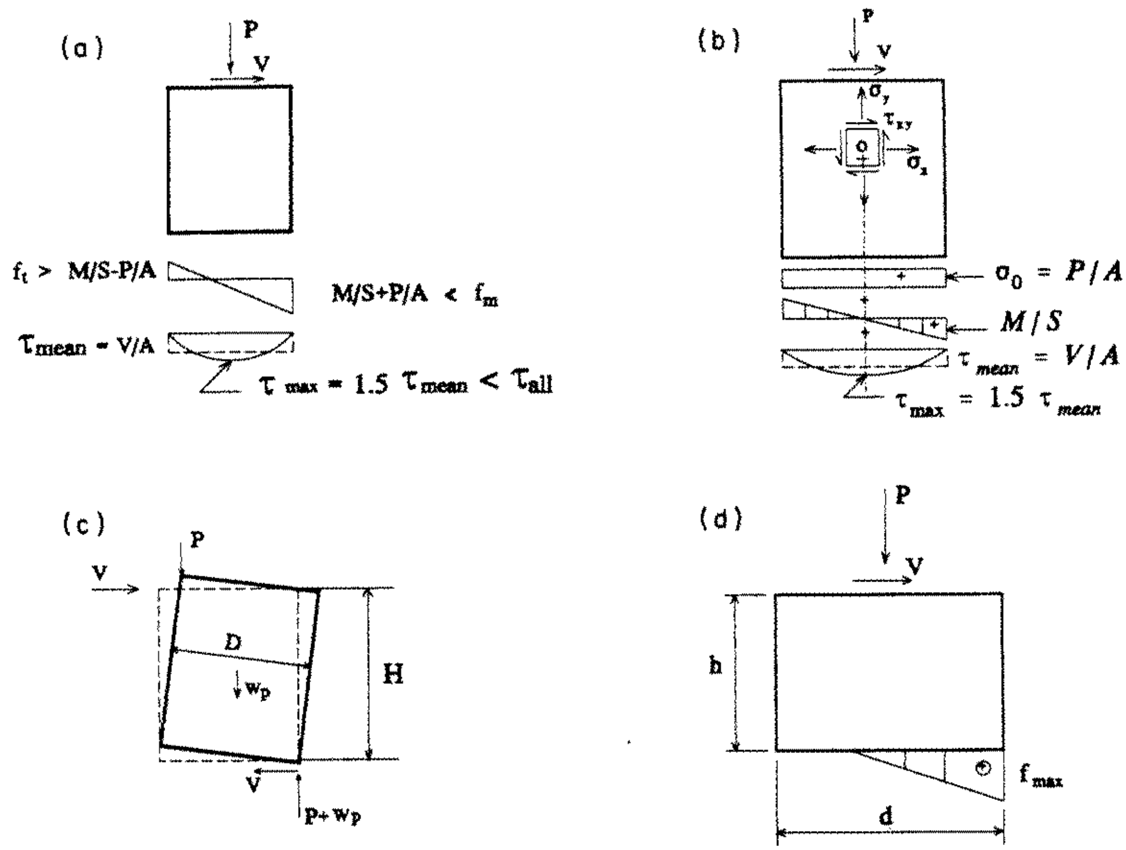

FIG. 12. Hand-calculation-oriented models of the in-plane response of unreinforced masonry walls: (*a*) state of practice; (*b*) principal stresses at panel centre; (*c*) ABK's dynamic restoring shear formulations; (*d*) Epperson and Abrams linear compression model on uncracked base.

(Gere and Timoshenko 1984), and knowing that the principal tensile stress, σ_1, cannot exceed the tensile stress capacity, σ_t, of the masonry for a given magnitude of the applied mean compressive axial stress, σ_0, the peak value of the shear stress, τ_u, that can be sustained at the centre point of the unreinforced masonry panel (Fig. 12*b*) before cracking initiates is

$$[10] \quad \sigma_1 = \frac{-\sigma_0}{2} + \sqrt{\left(\frac{\sigma_0}{2}\right)^2 + (\tau_u)^2} = \sigma_t = \tau_{max} = 1.5\tau_{mean}$$

Rearranging these terms,

$$[11] \quad \tau_u = \sigma_t\sqrt{1 - \frac{\sigma_0}{\sigma_t}} = 1.5\tau_{mean}\sqrt{1 - \frac{\sigma_0}{1.5\tau_{mean}}}$$

The maximum lateral load, V, that could be applied to this panel would then be

$$[12] \quad V = \frac{\tau_u A}{1.5}$$

For clarity, it is worth re-emphasizing that τ_{max} (and correspondingly $1.5\tau_{mean}$) is the fundamental shear stress capacity of masonry in pure shear (i.e., a material property), whereas τ_u is the maximum shear stress that can be applied on a given panel already stressed by an applied gravity load. Obviously, in this procedure, the other limiting conditions of flexural cracking and compression crushing at the extreme fibres of the piers would again need to be checked independently.

Some researchers (Sinha and Hendry 1969; Mayes and Clough 1975) have recommended that both the usual Coulomb friction shear strength equation (i.e., the one present in North American codes) and the above formulation be checked: The first reflects that bond and friction between the mortar joints could potentially govern at low axial compressions, whereas for high bond mortar and (or) higher axial loads, only the second would be applicable. Recent experimental evidence (Magenes and Calvi 1992) indirectly confirms that [11] is of questionable accuracy when failure (under cyclic testing) develops through the mortar joints. In addition, Samarasinghe et al. (1981) cautioned that the Coulomb equation is reliable only for panel geometry having length (L) to height (H) ratios in the range $1.2 < L/H < 2.0$, as verified both experimentally and analytically.

3.7.2.2. Flexural rocking failure

The above formulations implicitly postulate that shear strength is exhausted at the onset of first cracking. This need not be the limiting condition under flexural cracking, considering, even statically, the stabilizing effect of axial loads. Equations have been proposed to assess the ultimate strength of flexurally cracked piers having reserve shear strength capacity. ABK (1984) recommended the following expression for the dynamic restoring shear capacity, V_R, of unreinforced masonry piers:

$$[13] \quad V_R = 0.9\frac{PD}{H}$$

where P is the axial load on the pier, and D and H are the pier width and height respectively, as shown in Fig. 12*c* (the pier's self-weight is neglected in this equation). The value of 0.9 is recommended based on experimental results. For walls having non-negligible self-weight, P_W, the alternative equation:

$$[14] \quad V_R = 0.9\frac{(P+0.5P_W)D}{H}$$

is more proper, where 0.5 is the smaller lever arm applicable to that wall's weight. The above two equations have been integrated into the CGSEEB, consistently with the other dynamic stability and ultimate behaviour models adopted there. Alternative equations have been suggested by others (Priestley 1985) with the assumption that the bearing zone resisting overturning is, at ultimate, under a uniform compression of $0.85f'_m$, with practically equivalent results.

In this case, the presence of an axial compression force plays a determinant role in the performance of unreinforced masonry walls — it contributes to their overall stability beyond flexural first cracking. ABK (1984) were apparently the first to experimentally investigate this behaviour, conducting static and dynamic in-situ tests on parts of an unreinforced masonry building scheduled to be demolished. In static tests, a stable rigid-body rocking motion that attempts to develop following first flexural cracking is restrained if a sufficient axial compressive load is present. The pier's lateral load resistance is greatly enhanced by this effect, and the compression capacity of the masonry in the uncracked bearing area becomes the limiting factor, unless overturning occurs.

Epperson and Abrams (1990) also studied the combined effect of shear, flexure, and axial compression, testing piers extracted from an existing 70-year-old unreinforced masonry building. Although the specimens were stocky, their predicted flexural cracking strength was less than their shear cracking strength. These experiments confirmed that, in spite of initial flexural cracking at a wall's base, if the gravity compressive load stabilizes a wall against overturning and the compressive strength of masonry is not exceeded at the wall's toe, the applied lateral load can be increased until diagonal cracking occurs. Incidentally, for walls with a height-to-width aspect ratio ranging from 0.6 to 0.9, flexural cracking occurred on average at approximately 60% of the wall's ultimate strength under lateral load. The recorded lateral load versus top displacement curve, under monotonic loading, was highly nonlinear.

Prawel and Lee (1990*a*) studied unreinforced masonry walls with larger aspect ratios in which only flexural failure would occur. In both static and dynamic shake-table tests, flexural failure ensued from the development of an initial crack at the base of the wall, rapidly propagating across the full width and allowing noticeable rigid-body rocking and sliding along the horizontal crack surface. The sliding friction dissipated energy and produced an apparent ductility (of 2.25 on average), all while preventing significant loss of capacity past first cracking.

Implied above is that resistance against compression crushing at the wall's toe was not exceeded. Epperson and Abrams (1990) suggested that, for walls flexurally cracked at their base, the compressive strength of masonry at the wall's toe could be reached before overturning and diagonal tension (shear) failures. Assuming a linear distribution of stresses over the uncracked portion of the wall base, as illustrated in Fig. 12*d*, and neglecting masonry's tensile strength, the

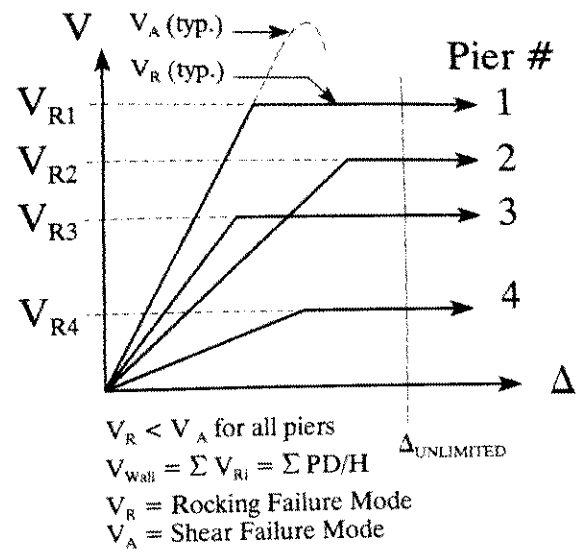

FIG. 13. Calculation of rocking failure mode resistance for a group of walls or piers (adapted from Lambert 1991).

expression derived from statical equilibrium to check for this maximum compressive stress at the wall's toe is

$$[15] \quad f_{max} = \frac{2P}{3b\left(\frac{d}{2} - \frac{Hh}{P}\right)} \le f'_m$$

where P is the sum of the vertical compressive force acting on the wall; b, d, and h are the wall's thickness, width, and height respectively; H is the applied horizontal force; and f'_m the ultimate compressive capacity of masonry. An adaptation of that result into a design format illustrated that the ultimate capacity of unreinforced masonry walls failing in flexure could be as high as three times that given by current codes, and ductile instead of brittle (Abrams 1992). An equation was also proposed to consider reductions in shear strength attributable to the loss of effective wall area as a result of flexural cracking.

Finally, in the absence of axial compressive forces, unreinforced masonry piers with extremely large height-to-width aspect ratios would behave brittle linear elastic in bending, i.e., first cracking coincides with complete failure, as verified by Davidson and Wang (1985).

3.7.2.3. CGSEEB masonry wall resistance

To provide a reliable assessment of the seismic resistance of an existing unreinforced masonry building, the shear resistance of its material must be measured on-site. This requires the use of procedures much different from those used in traditional laboratory masonry testing, and consequently, the resulting shear strength equation used in the seismic-adequacy assessment will be different as well. The procedure for the CGSEEB-recommended in-place shear test (also called push test) is described at length in that document, and need not be repeated here. The adopted equation for the shear strength of existing masonry walls is

$$[16] \quad v_m = C_1(C_2 v_t + P/A) = 0.56v_t + 0.75P/A$$

where C_1 is a calibration constant taken as 0.75; C_2 is a reduction factor used to adjust test values in order to account for the probable and unavoidable bounding effect of the

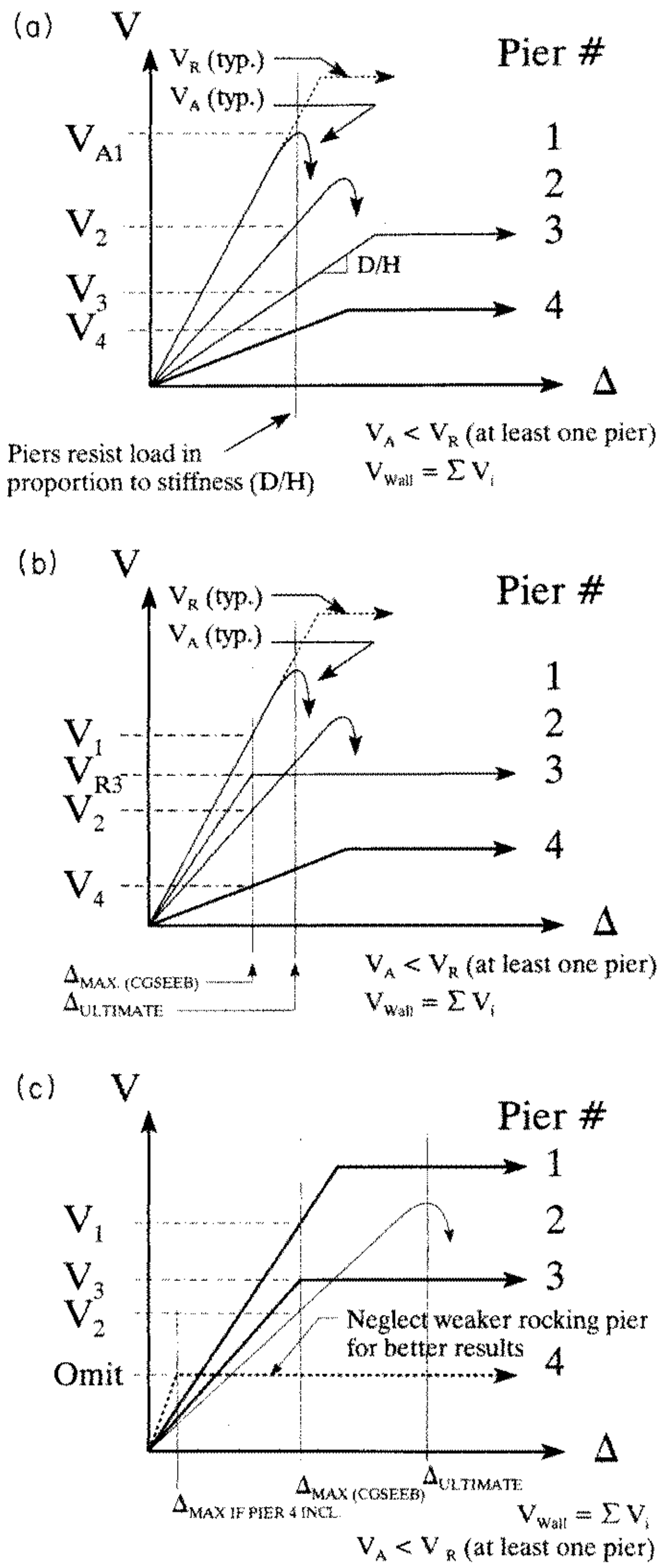

FIG. 14. Calculation of resistance for a group of walls or piers having a mix of rocking and shear failures where all walls or piers resist loads in proportion to their stiffness and (*a*) shear failure in at least one pier occurs first, (*b*) rocking failure in at least one pier occurs first, and (*c*) a much weaker rocking pier is neglected from calculations (adapted from Lambert 1991).

collar joint during the in-place shear test, and also taken as 0.75; v_t is the basic bed-joint shear stress which is exceeded by 80% of all the on-site test values when the effect of axial stresses is discounted; and P is the superimposed dead load at the top of the pier or wall under consideration. The value of C_1 was obtained by correlating together in-place test results, full-size on-site tests of walls, and finite-element analyses results. It partly reflects adjustments needed to account for the variability of workmanship. The conservative 20-percentile value of v_t reflects the fact that shear failure is initiated at the weakest flaw in a wall (ABK 1984; Asakura 1987; Kariotis 1989). The CGSEEB requires that unreinforced masonry be removed or pointed and retested if the obtained v_t is less than 0.2 MPa. Furthermore, v_t should not be taken as greater than 0.7 MPa in [16]. As obtained from strength of material principles, shear resistance for a wall failing by diagonal shear cracking is

$$[17] \quad V_A = v_m Dt/1.5$$

where D and t are the width and thickness of the wall. As discussed earlier, the CGSEEB also recognizes that walls can also rock prior to reaching their limiting shear stress, as defined by [13] and [14].

Finally, for a group of walls or piers acting as a single end-wall at a given level, a methodical strategy is provided by the CGSEEB to determine the global behaviour of the group. Conceptually, three cases are possible (Lambert 1991):

(*i*) If all walls or piers can develop a rocking mode, V_R, prior to reaching their shear-stress failure, V_A, as illustrated in Fig. 13, then a stable rocking mechanism for the end-wall group is possible, with the total force distributed to each wall or pier in proportion of their rocking strength (i.e., PD/H). Moreover, considering that the restoring force–displacement relationship dissipates energy in a stable manner, the sum of the rocking strengths need be able to resist only 60% of the applied load, V_{WX}. The upper limit of stable rocking behaviour, reached whenever shear failure occurs because of flexural cracking-induced loss of effective area (Abrams 1992) or when overturning of the wall or pier occurs, is not considered by this methodology to be within the practical range of seismic response.

(*ii*) If the shear resistance of at least one wall or pier is less than its rocking resistance, then the total shear force applied to the end-wall group must be shared by all resisting structural elements in proportion to their D/H ratio, provided that no element is loaded beyond its shear (V_A) or rocking (V_R) resistance. This is illustrated in Figs. 14*a* and 14*b*, the latter depicting an inherent conservativeness to this approach when piers start to rock at a lesser displacement than needed to produce shear failure of any other pier. The approximate stiffness distribution (D/H) assumes that shear deformations in the masonry wall or pier dominate their response (ABK 1984; SEAOC 1986).

(*iii*) As an extension of case (*ii*), and taking advantage of the stability of the rocking failure mode, any wall or pier whose calculated share of the shear force loading the end-wall exceeds its rocking resistance could be neglected in a subsequent reanalysis. As illustrated in Fig. 14*c*, this could prove advantageous in situations where a few much weaker rocking-capable piers are present.

A flow chart of this decision process is also available elsewhere (SEAOC 1986; Seismic Safety Commission 1990; Lambert 1991).

3.7.2.4. Shear-ductile failures

A large number of static tests have been carried out on unreinforced masonry shear walls in the past, as reported by Page et al. (1982). Most of these experimental investigations were aimed at establishing or verifying analytical failure equations, often those of the interaction of shear and axial stresses under static monotonically applied loads more representative of winds than earthquake loadings. Consequently, in most design codes and standards, including the CGSEEB when rocking failure does not prevail, a brittle failure is generally anticipated once ultimate shear stress resistance is exceeded. However, there is evidence that seismic-induced shear failures can also be ductile under some conditions.

Mengi and McNiven (1986) performed shake-table tests of an assembly of two parallel walls repeatedly subjected to the 1940 El Centro earthquake scaled to a progressively increasing intensity, from an excitation level producing linear elastic response to one where the walls could not sustain any additional damage. They observed that first cracking alone was not a realistic shear failure criterion under dynamic loading for this particular specimen. A post-cracking stable behaviour was found to exist until lateral deformations reached twice those corresponding to first cracking; this required a 38% increase in the peak ground acceleration of this earthquake record. Damaged walls were found to be still capable of elastic behaviour, albeit with a reduced secant shear modulus, G, and an increased damping coefficient, ξ. An ensuing equivalent linear model using these modified properties will be reviewed in a later section.

Recent tests by Konig et al. (1988) studying the in-plane post-cracking dynamic cyclic behaviour of unreinforced masonry shear walls demonstrated that, when cracking passes by the bed joints in a diagonal jagged pattern across the wall, the separated portions of the wall can slide onto each other, resulting in large relative deformations (with ductilities of up to 4) and little strength degradation (since this ultimate capacity is governed by friction resistance which remains nearly unchanged) before failure. For normal bricks, this type of failure occurs if the axial load is low; it may also occur under moderate axial loads if the ratio of the masonry strength to mortar strength is higher, as would be the case with solid masonry units. A typical hysteretic plot of the wall top displacement under cyclic shear forces is presented in Fig. 15. Under higher axial loads, the friction resistance of the bed joints is proportionally increased, and cracking occurs instead through the masonry units if the principal stresses locally exceed the tensile strength of the units. As a result, the individual separated portions of the walls tend to slide, with little apparent ductility, downwards along the more regular diagonal cracks. Finally, under very high axial loads, the resulting large compressive principal stresses in part of the wall lead to an explosive-type failure before the occurrence of any shear cracking, and obviously without any plastic deformability. These tests, as well as some others recently reported (Magenes and Calvi 1992), also validated a previously suggested analytical model of this shear strength dependence on the magnitude of the axial loads (Mann and Muller 1982).

Finally, results reported by Abrams (1992) further confirm the existence of hysteretic shear behaviour in unreinforced masonry walls, with stable and extremely large deflections (more than seven times those observed at peak load) attributable to considerable sliding (up to 25 mm for some bricks) along the bed joint.

3.8. Anchorage

In many unreinforced masonry buildings, there is a total absence of positive anchorage of the floors and roof to the unreinforced masonry walls. In the absence of positive anchorage, the exterior walls behave as cantilevers over the total building height. The risk of wall out-of-plane failure due to excessive flexural stresses at the base of the wall obviously increases with its height, but more importantly, global structural failure can result from the slippage of the joists and beams from their supports. Moreover, joists-to-walls anchors, when present in unreinforced masonry buildings, are of many different types, and their presence is often unrelated to seismic concerns. Thus, anchor failure is equally likely. While the metal of the anchor may fail, rupture may also occur at the connection points, i.e., the anchor could shear loose from the framing member at one end, or be pulled off the masonry at the other end. The details of these modes of rupture obviously vary with the type of anchors used.

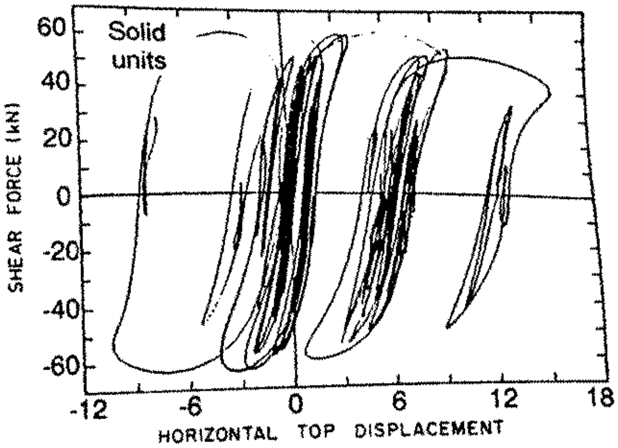

FIG. 15. Typical hysteretic curve of applied in-plane shear force versus top wall displacement, for cracking occurring in mortar only (adapted from Konig et al. 1988).

Essentially, anchors must be shear resistant in order to solidly connect the diaphragm to the end-wall, and tension resistant in order to anchor walls to the diaphragms in their out-of-plane excitation direction. Based on the aforementioned description of the structural behaviour of unreinforced masonry buildings, it is clear that the minimum shear connection force to transfer should be the lesser of

[18] $V_s = v'W_d/2$ and $V_s = v_u D$

and the minimum tensile anchor strength should be

[19] $V_P = 2.5v'W_n$

where W_n is the tributary weight of connected masonry, calculated from the mid-height of the storey above to the mid-height of the storey below. These requirements, as well as others that would ensure an overall good distribution of anchors, are included in the CGSEEB. Finally, because of the risk of "loosing" anchors embedded in the opening crack plane of walls rocking out-of-plane, through-wall anchors with plates are to be used whenever possible (Asakura 1987).

3.9. Final observations on the special procedure

It is tacitly understood that, as a trade-off for economic feasibility, the special procedure has been developed to provide a reduced design level and a higher risk of failure. However, this has been done in a context where structures which would otherwise have been left unaltered had to undergo a seismic evaluation in order to comply with special seismic hazard mitigation ordinances. This situation does not yet exist in the Canadian context, where buildings that are to be seismically upgraded are usually already undergoing upgrading work to palliate other deficiencies or to enhance their market value (i.e., overall rehabilitation work). The engineer should therefore, whenever possible, also consider the economics of providing seismic resistance in compliance with more stringent standards. However, since for many historical buildings alteration of the heritage fabric must be minimized, the above special procedure provides a viable alternative. In such situations, it is worthwhile that the ABK testing program established that the margin between the seismic intensities needed to initiate cracking and those needed to produce dynamic instability can be large enough to have a major impact on engineering decisions.

As with other normal engineering procedures, when the existing conditions are found to violate the tolerable limits, the structure must be retrofitted using a strategy of the structural engineer's choice. The CGSEEB redirects the engineer toward specialty documents broadly devoted to seismic retrofitting techniques (FEMA 1992*b*). However, for some particular deficiencies, the CGSEEB methodology explicitly mandates the nature of the corrective measures. In high seismic-risk zones, for example, if masonry piers provide bearing supports for steel beams, the CGSEEB calls for the addition of steel columns next to the piers; these will act as shoring of the gravity system should the unreinforced masonry piers fail.

Much additional information and examples are available for the City of Los Angeles Building Code and UCBC versions of the special procedure (SEAOC 1981, 1983, 1986, 1991). The reader is, however, cautioned that most of that information is in the working stress format, not the CGSEEB limit states design format. Finally, it is noteworthy that prior to the publication of the UCBC, a survey of consultant's practices (ATC 1987) regarding seismic evaluation of existing structures revealed that most structural engineers used nonstandardized methods largely based on their own experience or office practice, and used existing methodologies only if required to by the clients. It is believed that since that time this practice has changed, and that thousands of unreinforced masonry buildings in California have been upgraded according to the above procedure (or a part or variation thereof). The seismic performance of some fully or partially retrofitted buildings has already been reported (Moore et al. 1988; Deppe 1988; Rutherford and Chekene 1991; Bonneville and Cocke 1992). Anchorage of walls and parapets to roof and floors has already proven effective in substantially improving seismic performance, but the final verification of the adequacy of many other aspects of the special procedure in enhancing life-safety is still awaiting the test of a major earthquake there.

4. Modelling aspects of unreinforced masonry structures — beyond the CGSEEB

It is the prime responsibility of the structural engineer to establish, based on judgement and experience, a realistic structural model for a given unreinforced masonry building. Some engineers will reluctantly depart from proven, albeit conservative, traditional approaches suitable for hand-calculations, others will be inclined to try the special procedure provided by the CGSEEB. Aside from these, advanced modelling procedures such as dynamic analysis or finite element models are always possible. The individual engineer's confidence in the reliability of different approaches, along with the consideration of some nontechnical aspects such as liability and client sophistication, all influence modelling decisions, particularly in the absence of an "umbrella" ordinance prescribing a specific procedure. Yet the final judgement on the seismic adequacy of a given unreinforced masonry building is also very much dependent on the characteristics of the selected model. Some of the other approaches proposed in the literature are briefly reviewed in what follows, although they have not yet led to systematic methodologies.

4.1. Out-of-plane modelling of alternative boundary conditions

To date, research on dynamic stability has concentrated on walls implicitly modelled as continuous vertical slabs supported between floor levels (ABK 1984; Prawel and Lee 1990*b*) or only at their base (Bariola et al. 1990). The natural "vertical-anchorage" provided by continuity with the other perpendicular walls has been conservatively neglected. This continuity may significantly enhance the out-of-plane resistance of narrow walls. The effect of various boundary conditions on the ultimate out-of-plane strength of unreinforced masonry panels, in a nonseismic context, has received some attention: a fracture-line model which is applicable to orthotropic brickwork panels of low tensile strength, proposed by Sinha (1978, 1980), is noteworthy — it is not to be confused with the well-known yield-line theory for the analysis of reinforced concrete slabs, although it is conceptually similar. In this upper-bound approach, lines of maximum moment and zero shear are assumed to be linear; the cracking moment is assumed to be reached simultaneously along these lines, with ensuing rigid-body rotation of the separated unreinforced masonry panel portions, and then failure. The panel's boundary conditions on all sides, for any arbitrary geometry, are directly considered by the method. As with all upper-bound methods, the maximum capacity of the panel is determined by finding the path of the fracture lines, which allows failure under the lowest magnitude of applied loads. While Sinha found an excellent correlation with experimental results, the unreinforced masonry panels tested were free of concurrent in-plane axial loads. An adaptation of this method into a dynamic stability framework has not yet been attempted.

4.2. Determination of dynamic characteristics

Among the few studies that report on the experimentally measured damping properties of unreinforced masonry, there is no clear consensus. Benedetti and Benzoni (1984), in their study of stone masonry, calculated values of damping equal to 4% and 12% of the critical damping by Fourier transforms of the response, for linear low-intensity and nonlinear high-intensity tests respectively. Prawel and Lee (1990*a*, 1990*b*) calculated the secant stiffness and secant damping coefficients from banded white-noise tests and spectral analysis for the in-plane and out-of-plane behaviour of walls with various levels of damage. In both cases, they observed a rapid degradation of stiffness immediately following first cracking, coupled with a proportional increase in damping (on average 4% to 13% in-plane, 4% to 8% out-of-plane). Jiugqian and Maogong (1984) also observed a sudden drop in stiffness, of up to 60% of the original value following first cracking; their recorded damping values ranged from 1% to 5%, under low and high axial compressive stresses respectively.

In ABK's aforementioned in-plane test of unreinforced masonry piers (ABK 1984), dynamic free-vibration response and the apparent viscous damping coefficients under rocking action were also measured following the release of pre-imposed displacements. Values of 3.5% to 7.5% of critical damping were recorded.

4.3. *Finite element models*

Linear elastic finite element analyses are becoming popular, particularly in Europe, to establish the state of stress in complex unreinforced masonry heritage structures, which are often built of stone (e.g., Pistone et al. 1991). At this time, only a few reported studies are concerned with seismic resistance (Elsesser et al. 1991; Quirós Lara and Gutiérrez 1991; Vestroni et al. 1991; Karantoni and Fardis 1992; Magenes and Calvi 1992; Tena-Colunga and Abrams 1992); most investigate other loading conditions or general structural distress. These linear elastic analyses may be useful in providing some guidance as to the governing failure mode, ultimate elastic capacity, natural frequencies, mode shapes, and modal participation factors of uncracked unreinforced masonry buildings, but they provide limited insight into the ultimate strength and seismic behaviour of such structures; the method also produces unavoidable local stress concentrations which require careful interpretation.

Recognizing these limitations, some researchers have investigated the adequacy of special nonlinear and cracking finite elements in studying the ultimate seismic behaviour of structures. Both discrete-crack and smeared-crack formulations have been tried.

In the discrete-crack model, a special interface element (also called a gap, or gap contact, element) is introduced to allow separation of adjacent elements when the tensile strength of the material at this interface is exceeded. The layered structure of masonry suggests modelling the mortar joints as nonlinear gap elements, and the bricks as standard four-node elastic isotropic elements.

Chiostrini et al. (1989) adopted this approach in order to analytically replicate the ultimate behaviour of a masonry panel monotonically tested in shear. Their gap element consisted of two parallel plane surfaces capable of separating or sliding to break the bond between adjacent brick elements; only compressive normal stresses, by contact, and shear stresses, by Coulomb's friction, can be transferred across this element. The judicious selection of the friction coefficient is partly responsible for the reported excellent agreement with experimental results in which, incidentally, damage was confined to the mortar joints. Obviously, it is neither practical nor desirable to model all bricks and mortar joints in order to study the seismic response of entire unreinforced masonry buildings. Thus, in a companion study, Chiostrini and Vignoli (1989) used macro-masonry elements, in combination with gap elements, to model flexurally dominant cracking modes of piers or spandrels, as well as global overturning; these ultimate collapse mechanisms could be well replicated analytically, and their capacities compared satisfactorily with those predicted using other means. However, the discrete-crack strategy assumes prior knowledge of ultimate behaviour so that the gap elements can be adequately located in the model, i.e., where cracking will occur (Chiostrini and Vignoli 1991; Barberis et al. 1991). Results are otherwise unreliable. The model is also unable to model cross-brick cracking.

Alternatively, smeared-crack models were studied. In this model, the nonlinear effect produced by the opening and closing of cracks is integrated into the element formulation itself. Essentially, when the tensile strength of the material is exceeded, as determined by the internal state of stresses in a given element, cracking occurs orthogonally in relation to the tension principal stress and is assumed to be uniformly distributed (i.e., smeared) over the whole element. Clearly, meshing refinements in the zones of cracking improve the reliability of the solution.

However, reported finite element investigations of the behaviour of unreinforced masonry structures using smeared-crack models remain few and, again, mostly concerned with gravity loads. In one of the studies, which addresses the seismic response of unreinforced masonry walls (Chiostrini and Vignoli 1991), the predicted ultimate response and dynamic properties of a wall modelled using plasticity and smeared-crack elements are compared, and some theoretical developments are presented regarding the addition of geometrical and material nonlinearity capabilities to the smeared-crack element, all aimed at the study of slender unreinforced masonry components. In another study (Shing et al. 1992), it is reported that special interface elements used with smeared-crack elements can realistically replicate the brittle shear failure of unreinforced masonry walls and influence of mortar joints, provided that an a priori knowledge exists of the location and orientation of the critical cracks. More research is needed in order to fully assess the potential of this finite element strategy.

The ability to computationally predict the seismic response of unreinforced masonry structures will obviously improve in relation to advances in the modelling of the masonry material itself through finite element analyses. For example, research (Ignatakis et al. 1989) in which masonry units and mortar joints are modelled separately, each with its own tridimensional nonlinear material characteristics, has been successful in replicating experimentally obtained ultimate loads, displacements, damage pattern, and failure mode of unreinforced masonry panels subjected to in-plane loading. However, since each brick and adjacent mortar bed are individually meshed by up to eight triangular elements, the approach is obviously better suited to research in fundamental behavioral characteristics than to the study of complete structures.

Innovative techniques are also being investigated. For example, in one case, a new type of no-tension fan-shaped finite element has been proposed (Braga and Liberatore 1990) to improve the computational efficiency over traditional meshing approaches. The element was specifically developed to analyze the seismic response of masonry structures, but no results from this particular research endeavour have been reported at the time of this writing. The boundary element method (Brebbia and Niku 1991) and the modified distinct element model (Morales and Delgado 1992), proposed alternatives to the finite element method, are also currently being considered by some researchers for the study of unreinforced masonry buildings, but the strength of the former method resides largely in linear elastic analyses and crack propagation studies under monotonic loading, while the latter, which seems promising, has already proven useful in identifying general trends and behaviour.

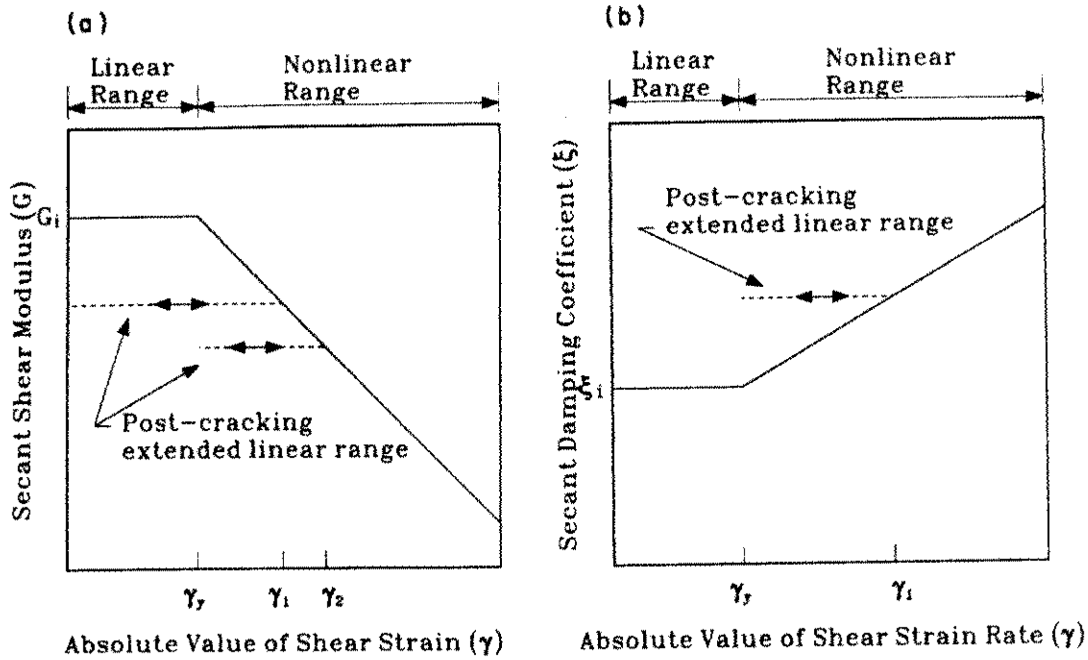

FIG. 16. (*a*) Secant shear modulus model and (*b*) secant damping coefficient model (adapted from Mengi and McNiven 1986).

Finally, as a word of caution, it is important to realize that none of the above finite element analysis strategies seems to have matured to the level of sturdiness and reliability required to be readily used by practising engineers. In particular, mesh objectivity is difficult to achieve, and numerical stability problems are known to exist when using discrete or smeared-crack models.

4.4. Hysteretic models for unreinforced masonry buildings

A comprehensive hysteretic model which captures the seismic behaviour of masonry while considering its heterogeneous nature, with its inherent and complex interaction of mortar, brick, layering patterns, and other distinctive features, remains elusive. As practising engineers have conventionally treated masonry as a linear elastic homogeneous isotropic material whose properties are obtained from standard static tests, most researchers are also comfortable with this simplified model. Yet some attempts at developing more advanced hysteretic models to capture the aforementioned experimentally observed nonlinear behaviour are noteworthy.

Mengi and McNiven (1986) proposed an equivalent linear model that accounts for the nonlinearity effect through variable secant shear modulus and secant damping coefficient. As one might expect, beyond first cracking, the shear modulus progressively decreases as a function of the shear strains whereas the damping coefficient increases on account of accrued friction-slip within the cracks. This is conceptually illustrated in Fig. 16. Bilinear relationships between the values of the secant shear modulus and the secant damping coefficient as a function of the absolute values of the shear strains and shear strain rates respectively were found to be adequate. However, this model should be used with caution. Generally, computer programs capable of conducting seismic nonlinear inelastic analyses of structures clearly differentiate damping from hysteresis; the damping coefficients relate to the elastic damping only. The notion of increased damping to simulate hysteretic behaviour should not be misused. In Mengi and McNiven, both G and ξ are evolving variables, with a re-linearization to new values being performed as new cracking occurs throughout given time-history analyses; incorrectly adopting constant values for use in an equivalent linear elastic analysis would provide neither a safe nor realistic assessment of the nonlinear inelastic seismic performance. Mengi et al. (1992) developed a computer program to implement the above model for special types of residential unreinforced masonry structures with rigid diaphragms, but its validation has been limited. Moreover, as for the analysis of a complete North American building, it is unknown whether the calculation of the secant damping coefficients of each component and their subsequent assembly into a global structural damping matrix would yield the correct results. Nonetheless, some researchers (Prawel and Lee 1990*a*, 1990*b*) have already reported on the experimentally obtained values of these secant properties.

A true hysteretic model applicable to unreinforced masonry has been proposed by Benedetti and Benzoni (1984). Developed in order to replicate experimentally obtained shear stress–strain hysteretic curves, it is constructed from three superimposed bilinear hysteretic shear subelements, which fail brittly at a prescribed strain intensity (Fig. 17). Each subelement is completely defined by its elastic shear modulus, G_i; limit elastic shear stress and strain, τ_i and γ_i respectively; and ultimate limit shear strain, γ_{iL}. The resulting shear-panel hysteretic curve is built from the direct summation of all individual subelements' strength, but subelements having exceeded their limit strain, γ_{iL}, are permanently removed from the model, as shown in Fig. 17. An estimate of the overall post-cracking "ductile" behaviour is given by the ratio γ_{1L}/γ_3. The value of the maximum shear stress, τ_{max}, depends on the average applied normal stress, σ, and critical tensile strength, σ_t, by the relationship of [11] presented earlier. The other parameters shaping this phenomenological hysteretic envelope are calibrated from available experimental results. While Benedetti and Benzoni did not provide a physical mechanism to obtain these other parameters without the need for experimental calibration, they

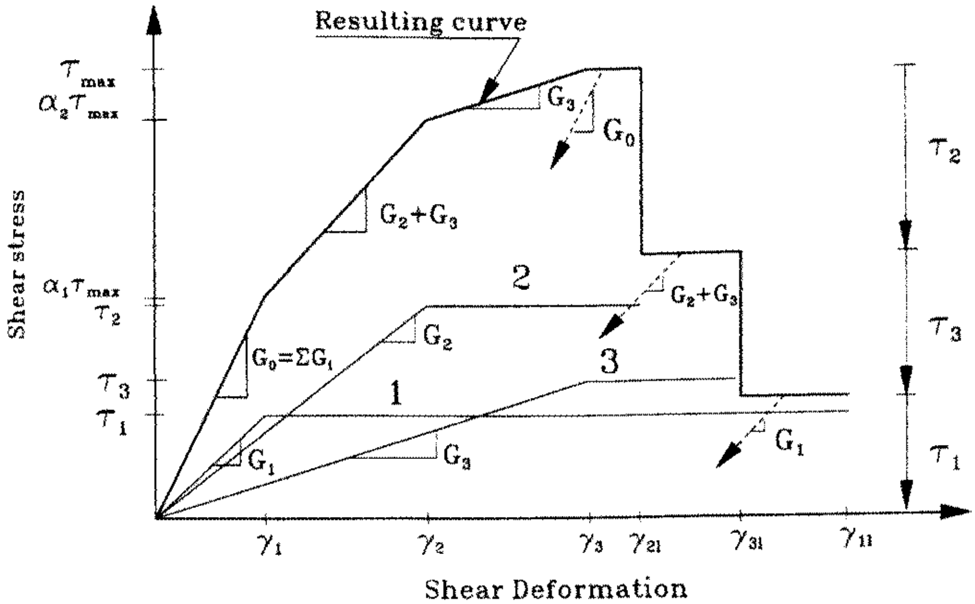

FIG. 17. Hysteretic shear stress–strain model proposed for unreinforced masonry (adapted from Benedetti and Benzoni 1984).

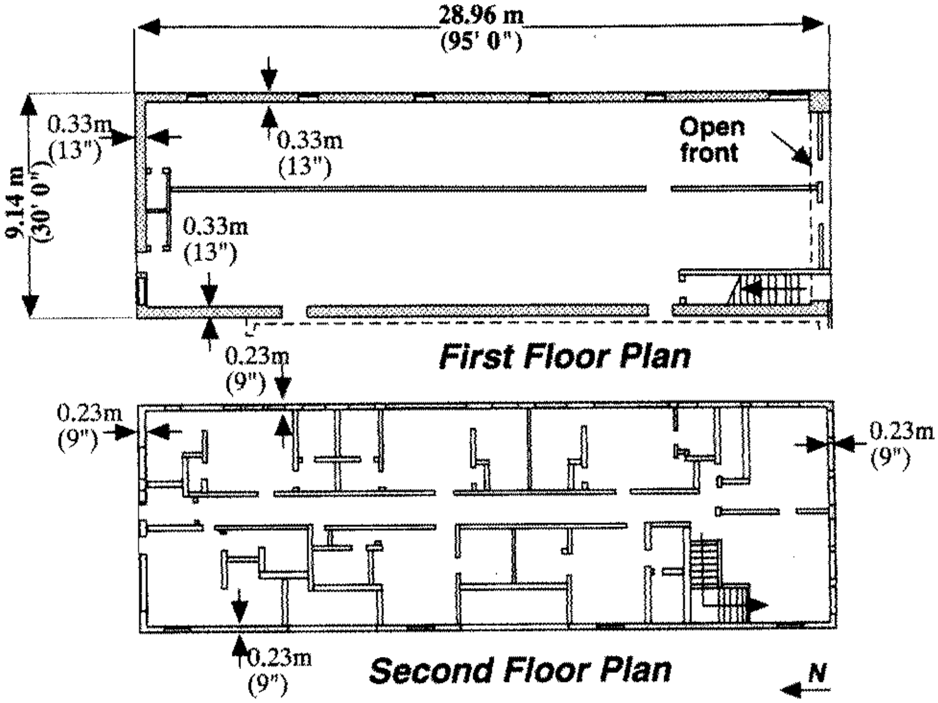

FIG. 18. Floor plan of example building.

demonstrated that the parameters α_1, α_2, (defined in Fig. 17), and τ_{max} have the most dominant influence on the nonlinear inelastic behaviour. Also, other structural elements must be used jointly to provide axial load resistance, as this hysteretic model only simulates a shear-panel response mechanism. Finally, as this model appears to have been derived from tests on stone masonry, its applicability to general unreinforced masonry structures is only inferred.

5. Example of the CGSEEB special procedure

At this time, it is clear that the CGSEEB special procedure is likely the most readily usable method for the seismic evaluation of unreinforced masonry buildings; a short example to illustrate its applicability follows. For the sake of brevity, clauses, figures, and tables contained in Appendix A of the CGSEEB document will not be repeated, but are cross-referenced as needed.

The two-storey building selected for this example is identical to the one analyzed by Breiholz (SEAOC 1983, 1986, 1991). It is 9.144 × 28.96 m (30 × 95 feet) in floor plan (Fig. 18) (converting into metric units the dimensions of old existing buildings unfortunately results in these awkward numbers). It has unreinforced masonry bearing exterior walls three and two wythes thick at the first and second storeys respectively, with corresponding thicknesses of 330 mm (13 in.) and 230 mm (9 in.). Normal density bricks are used.

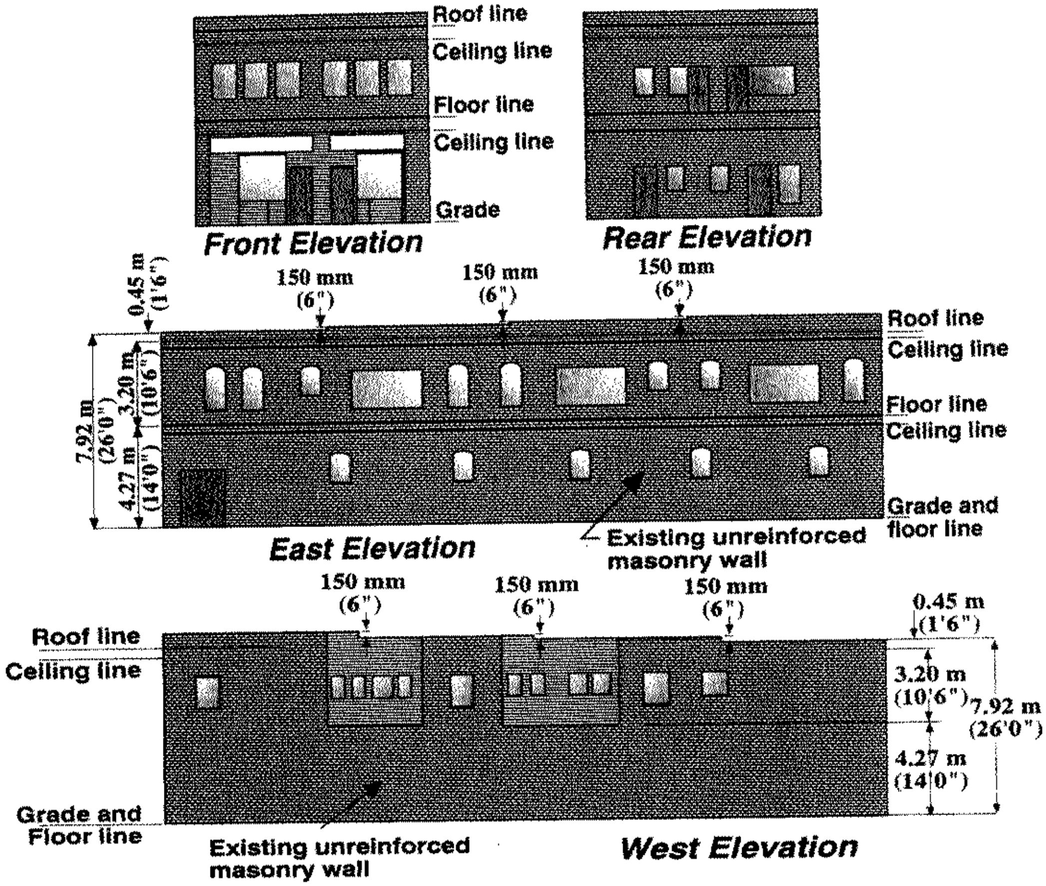

FIG. 19. Wall elevations of example building.

All walls have a number of window and door openings, and the south wall first storey is an open facade as is often the case for this type of building (Fig. 19). The roof is made of 13 mm (0.5 in.) plywood sheathing over straight sheathing, while the floor is 13 mm (0.5 in.) plywood sheathing over diagonal sheathing and finished wood flooring. The ground floor is a nonstructural slab on grade. At the roof level, the nonmasonry dead loads consist of roofing (0.22 kPa), sheathing (0.14 kPa), framing members (0.14 kPa), ceiling (0.48 kPa), and the tributary weight of wood partition (0.36 kPa), for a total of 1.34 kPa. At the floor level, these dead loads are floor finish (0.10 kPa), sheathing (0.24 kPa), framing members (0.17 kPa), ceiling (0.38 kPa), and the tributary weight of wood partition (0.72 kPa), for a total of 1.61 kPa.

This building can be analyzed using the special procedure of the CGSEEB (Appendix A), since it meets the conditions stated in Clause A.4 of that document; namely, it possesses flexible diaphragms at all levels above its base, has less than 6 storeys (for brick masonry), relies solely on unreinforced masonry walls to resist lateral loads, and has a minimum of two lines of such vertical elements parallel to each side of the building. Although the presence of an open front on one side violates the fourth condition above, the procedure recognizes this as a common condition in older unreinforced masonry buildings and, as will be seen later, has special provisions to address this.

In this example, it is assumed that the partitions shown on that floor plan are not positively connected to the underside of the top diaphragm, but rather stop at the level of a dropped ceiling. Hence, these partitions are not qualifying crosswalls (Clause A.11). Also assumed are the good quality of the masonry, the presence of an adequate number and configuration of headers courses in the wall lay-up (Clause A.5), and a mortar shear strength of 0.2 MPa, the minimum quality tolerated by the CGSEEB. Normally, the above information would be obtained from site inspection and shear tests of the type and quantity specified by Clause A.5.

This example is developed for the most severe seismic zone that can be encountered in Canada. Hence, the effective velocity ratio, v', defined by [1], is taken as 0.4, and the CGSEEB effective zonal ratio, Z', defined by

$$[20] \quad Z' = Z_v \text{ (NBC)} + 1 \text{ (if } Z_a > Z_v) + 1 \text{ (if } F \geq 1.5)$$

is taken as 6. The parameters Z_a, Z_v, and F in the above equation are defined in the NBCC (NRC 1990). Various abatements provided by the CGSEEB for the lesser seismic zones will be reviewed at the end of this example.

First, the total dead load, W_d, tributary to each diaphragm must be determined. The results of this calculation are summarized in Table 2. In agreement with the load-path model presented earlier, the inertia forces resulting from the tributary weight of head-walls at each level must be resisted by the end-walls, and included in each W_d. This masonry component of W_d is usually significant, and it is worth the extra effort to deduct wall openings in this calculation, as shown in Table 2.

The demand–capacity ratios, DCR, must then be calculated. Starting from the roof for earthquake excitations in the E–W direction, and using values prescribed by the

CGSEEB for the shear strength of the diaphragms, equation [2] of this paper (equation A-19 of the CGSEEB) gives

$$[21] \quad \mathrm{DCR} = \frac{2.5v'W_d}{\sum v_u D} = \frac{2.5(0.4)(749\ \mathrm{kN})}{2(4.4\ \mathrm{kN/m})(9.14\ \mathrm{m})} = 9.3$$

Figure 3 of this paper (or, equivalently, Fig. A-2 of the CGSEEB) reveals that the maximum demand–capacity ratio permissible for a diaphragm of 28.96 m span is 4.0. Thus excessive deformations of the existing diaphragms during an earthquake may produce out-of-plane failure of the head-walls.

Although the retrofitting of unreinforced masonry buildings is beyond the scope of this paper, it is nonetheless interesting to determine the needed strength and configuration of new crosswalls. Using [4] above (equation A-20 of the CGSEEB), it is determined that, to limit the demand–capacity ratio to 4.0, the total needed shear capacity of crosswalls, V_{cb}, is 107 kN. Therefore, assuming that a wall of nailed plywood on wood frame can be designed to develop a 13 kN/m shear strength, 8.23 m of such new crosswalls are needed. For good energy-dissipating action, it is necessary to ensure that crosswalls are well distributed along the whole length of the diaphragm, and capable of developing a reasonably high level of hysteretic energy when compared to the diaphragm's own inelastic behaviour. Hence, the CGSEEB requires that the sum of crosswalls' capacity over any 12.5 m be at least 30% of the shear capacity of the strongest diaphragm "at or above the level under consideration" (Clause A.11). As the current calculations are for the roof level only, the needed strength over 12.5 m is (30%)(4.4 kN/m × 9.16 m)(2) = 24.2 kN. The provision of four crosswalls of the aforementioned design, spaced at 5.75 m centre-to-centre, and each 2.25 m long, is adequate. However, since the shear strength of each crosswall (13 kN/m) exceeds the shear strength of the roof diaphragms that it connects (2 × 4.4 kN/m), collectors at least 1.1 m longer than the crosswalls must be designed to carry a drag force of 9.42 kN each. Examples of such collectors are presented elsewhere (FEMA 1992*b*).

The CGSEEB allows that crosswalls be present only between the roof diaphragm and the floor diaphragm immediately below it; implicitly, posts or columns must then carry the crosswalls' overturning moment through the storeys below. In this case, the super-diaphragm equation [5] can be used to calculate the demand–capacity ratio for the floor diaphragm below the roof level, but V_{cb} must be omitted since no diaphragm exists below the floor under consideration (equation A-22 of the CGSEEB). From this equation, a DCR value of 3.81 is obtained for the floor diaphragm, which confirms that no crosswalls are needed aside from those already provided above. For earthquake excitation in the N–S direction, the same equation [2] would give values of 1.94 and 0.47, much less than the maximum ratio of 5.0 permitted for a 9.14 m diaphragm span.

Using the above DCR values with Fig. 3 and Table 1, the out-of-plane dynamic stability of the unreinforced masonry walls is then verified. These calculations are summarized in Table 3. In this case, it is found that the second storey unreinforced masonry walls are likely to fail out-of-plane during an earthquake unless braced. Incidentally, for this building, the parapets are also too slender and need bracing (of a different type) as their slenderness ratio (h/t) ranges from 2.0 to 2.67, depending on location. Some effective

TABLE 2. Calculation of total dead load tributary to each diaphragm (W_d)

		Diaphragm			Head walls									
Earthquake action	Storey	Weight (kPa)	Area (m²)	W_{di} (kN)	Wall	Location	Weight (kPa)	h (m)	l (m)	Area (m²)	Size of openings (m²)	A_{NET} (m²)	W_{di} (kN)	W_d (kN)
E–W	Roof	1.34	265	355	East	Above roof	4.4	0.53	28.96	15.44	0	15.40	67.8	
						Below roof	4.4	1.60	28.96	46.34	15.33	31.00	137.0	
					West	Above roof	4.4	0.53	28.96	15.44	0	15.44	67.8	
						Below roof	4.4	1.60	28.96	46.34	18.58	27.76	122.0	749
E–W	Floor	1.60	265	424	East	Above floor	4.4	1.60	28.96	46.34	14.25	32.10	141.0	
						Below floor	6.3	2.13	28.96	61.68	12.00	49.70	313.0	
					West	Above floor	4.4	1.60	28.96	46.34	14.25	32.10	141.0	
						Below floor	6.3	2.13	28.96	61.68	6.00	55.70	351.0	1370
N–S	Roof	1.34	265	355	North	Above roof	4.4	0.61	9.14	5.58	0	5.58	24.6	
						Below roof	4.4	1.60	9.14	14.63	3.64	11.00	48.4	
					South	Above roof	4.4	0.46	9.14	4.20	0	4.20	18.5	
						Below roof	4.4	1.60	9.14	14.63	3.64	11.00	48.4	495
N–S	Floor	1.60	265	424	North	Above floor	4.4	1.60	9.14	14.63	1.43	13.20	58.1	
						Below floor	6.3	2.13	9.14	19.47	1.82	17.60	111.2	
					South	Above floor	4.4	1.60	9.14	14.63	3.64	11.00	48.4	
						Below floor	6.3	2.13	9.14	19.47	9.36	10.10	63.7	705

TABLE 3. Calculation of walls out-of-plane dynamic stability

Earthquake direction	Walls	Level	h (m)	t (m)	h/t	DCR	Span (m)	Region in Fig. 3	Allowable (h/t) (Table 1)	Needs bracing
E–W	East and west	Second level	3.7	0.230	16.1	4.00	28.9	2	14	Yes
	East and west	First level	3.7	0.330	11.2	3.81	28.9	2	16	No
N–S	East and west	Second level	4.3	0.230	18.7	1.94	9.1	3	9	Yes
	East and west	First level	4.3	0.330	13.0	0.47	9.1	3	15	No

TABLE 4. Calculation of in-plane applied loads

Storey	Wall	W_{WX} (kN)	W_d (kN)	F_{WX} – (eq. [6]) (kN)	v_u (kN/m)	D (m)	F_{WX} – (eq. [7]) (kN)	Min. F_{WX}	V_{WX}
Second	North	73	749	179	4.4	9.14	69.4	69.4	69.4
	South	67	749	177	4.4	9.14	67.0	67.0	67.0
	East	205	495	181	4.4	28.96	209	181	181
	West	190	495	175	4.4	28.96	203	175	175
First	North	169	1370	342	26.0	9.14	305	305	374
	South	112	1370	319	26.0	9.14	282	282	349
	East	454	705	323	26.0	28.96	323	323	504
	West	492	705	338	26.0	28.96	338	338	513

TABLE 5. Calculation of shear and rocking resistance of north wall

Pier	D (m)	H (m)	P_D (kN)	P_D/H (kN)	V_R (kN)	V_A (kN)	$V_R < V_A$ (kN)
			Second storey				
1	1.93	1.22	19.2	29.2	26.3	33.1	Yes
2	0.533	1.22	14.0	6.16	5.54	9.16	Yes
3	0.838	2.13	23.9	9.32	8.39	14.4	Yes
4	0.66	1.22	16.3	8.82	7.94	11.4	Yes
			First storey				
1	1.98	2.13	71.3	66.3	59.7	84.4	Yes
2	0.46	1.07	29.2	12.6	11.3	25.9	Yes
3	1.02	1.07	51.4	48.8	43.9	50.8	Yes
4	0.76	1.07	45.6	32.4	29.2	41.5	Yes
5	0.30	1.68	34.9	6.28	5.66	24.8	Yes
6	0.61	1.68	31.7	11.4	10.3	30.9	Yes

special bracing retrofit techniques are presented elsewhere (FEMA 1992*b*).

In-plane loading of the end-walls and their resistance must then be determined. In the absence of qualifying crosswalls at *all* levels, equations [6] and [7] are applicable (equations A-7 and A-8 of the CGSEEB). A calculation of the applied loads is presented in Table 4. Seismic resistance of the north wall (Fig. 20) is evaluated in Table 5 using equations [13], [16], and [17] (A-14, A-3, and A-13 of the CGSEEB respectively). It is noteworthy that although the gravity load contribution from the diaphragm can be included in the calculation of P_D, much of that weight in older unreinforced masonry buildings is often supported by columns and posts of an internal gravity-only structural system (e.g., timber posts, cast-iron columns). Moreover, timber floor framing systems often dominantly carry the tributary diaphragm gravity loads to only a few parallel walls. In this example, the selected building is assumed to exhibit such a bias, with the diaphragm's framing system supported by the west and east walls. Hence, in Table 5, P_D only accounts for the wall's masonry overburden. Some approximations and sensitivity analyses are also necessary in calculating P_D when openings are not of identical size and location in successive storeys.

For both storeys, it is found that rocking failure mode governs over the shear failure mode of all piers. Rocking failure mode of the top storey is schematically illustrated in Fig. 21. Hence, only 60% of the applied load need be resisted by each storey. For the second storey,

$$[22] \quad 0.6V_{WX} = 41.6 \text{ kN} < \sum V_R = 48.2 \text{ kN}$$

whereas, for the first storey,

$$[23] \quad 0.6V_{WX} = 224 \text{ kN} \nless \sum V_R = 160 \text{ kN}$$

This indicates that, for the north wall, in-plane strengthening is required at the lower storey only. Again, the reader is referred elsewhere for an overview of possible retrofit strategies (FEMA 1992*b*). Similar calculations can be performed for the other three walls. However, the first storey open facade of the south wall deserves particular attention. For this special case, the CGSEEB-proposed equivalent diaphragm equation (Clause A.12) is not applicable, since this is not a single-storey building (Clause A.4). Alternatively, the CGSEEB prescribes that a new steel or reinforced-concrete frame can act as the "missing" unreinforced masonry wall by (*i*) resisting 100% of the loads at that location; (*ii*) being stiff enough to restraint interstorey drifts to less than 0.0075 of the storey height; and (*iii*) coexisting with unreinforced masonry in that facade only if the few unreinforced masonry piers present fail by rocking, not by shear. The design of such a steel frame is otherwise conventional and not covered here.

Finally, the entire CGSEEB procedure relies on the proper anchorage of walls to diaphragm, as per equations [18] and [19] (A-4, A-5, and A-6 of the CGSEEB). In this particular example, anchors must be provided to resist shears of 4.4 and 26 kN/m respectively at the roof and floor levels of both the north and south walls, and 3.42 and 4.87 kN/m respectively at the roof and floor levels of both the east and west walls. Moreover, anchors must be provided to resist tension

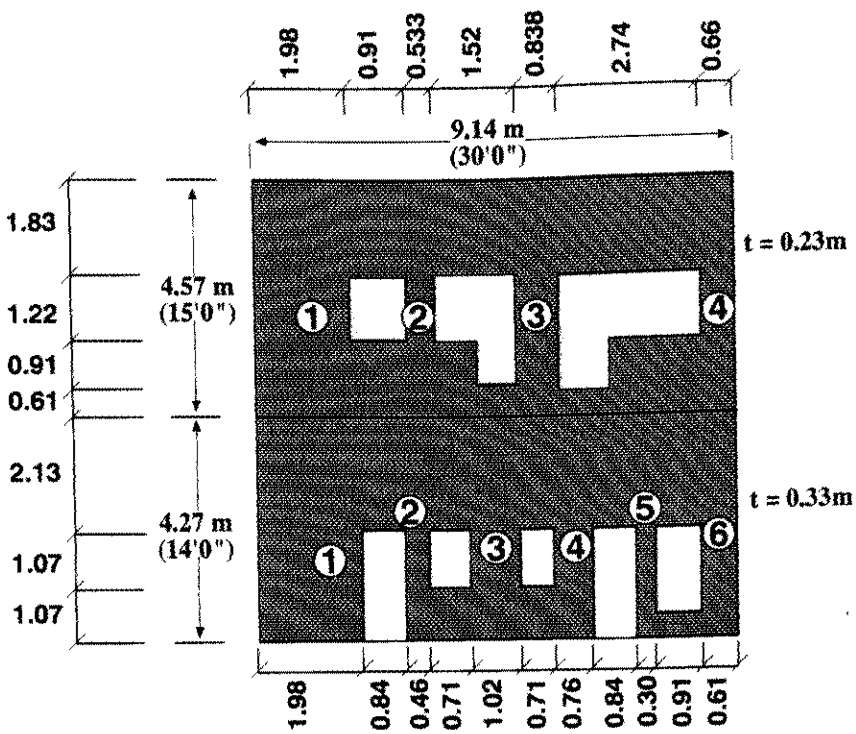

FIG. 20. Dimensions and piers considered for north wall in-plane strength evaluation.

forces of 9.72 and 20.46 kN/m of wall at the roof and floor levels respectively. Obviously, it is advantageous to select anchors which can simultaneously resist both types of excitation. Calculations demonstrate that 19 mm diameter through-wall bolts centred in 64 mm diameter holes filled with nonshrink grout (so that the shear bearing pressure of bolts does not crush the masonry), at 0.5 m spacing, each with at least a 20 000 mm^2 bearing plate, are adequate everywhere. According to the CGSEEB, these can develop a combined strength of 13 kN/m in shear, and 12 kN/m in tension when anchored in a two-wythe wall. For this example and in most similar cases, it must be appreciated that in determining reliable wall thickness, changes in wall thickness usually occur just below the floor diaphragm, since the thicker wall also often acts as a support for the floor framing system. Also, in this case, a single anchoring scheme is used for simplicity and to minimize the risk of errors during construction. The chosen spacing also automatically insures that anchors are provided close to the inside corners of walls; the flanges of corner walls are frequently neglected in order to simplify calculations, and their rigidity is therefore higher than considered and will attract more loads than predicted by calculations. These choices also respect the maximum spacing requirements of the CGSEEB. For aesthetic concerns, the bearing plates should be chosen as architectural decorative elements or, alternatively, should be of a discrete colour.

As mentioned earlier, if the building in this example was located in a region of lesser seismicity, fewer elements of the unreinforced masonry building would be subject to evaluation. For example, the CGSEEB does not require any seismic evaluation for unreinforced masonry buildings in effective seismic zones of 1 or less (i.e., where $Z' \leq 1$). In zone 2, it requires only that masonry be of sufficient quality, that wall anchorage be adequate, and that out-of-plane dynamic stability of parapets be ensured. Dynamic stability of all walls, in-plane wall strength, and diaphragm shear transfer need only be verified for buildings located in effective seismic zones 3 or higher. Demand–capacity ratio calculations are not required unless in zone 5 or above. Moreover, the out-of-plane dynamic stability criterion is also relaxed in less severe seismic zones, as shown in Table 6. However, while the seismic velocity excitation at the edge of the diaphragms is reduced almost proportionally with seismic zone, the diaphragm dynamic velocity amplification increases as the diaphragm's behaviour becomes more elastic. The interplay of these counter-balancing effects explains the apparently slow reduction in the maximum permissible wall slenderness to ensure dynamic stability.

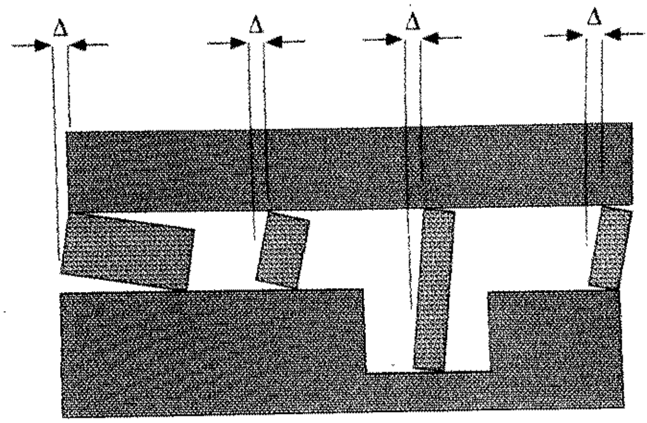

FIG. 21. Schematic illustration of rocking failure mode, second storey, north wall.

6. Conclusions

Although masonry is a brittle material incapable of inelastic straining, there are energy dissipation mechanisms available in standard unreinforced masonry construction which can

TABLE 6. Wall slenderness limits to ensure out-of-plane dynamic stability in various seismic zones

	Effective seismic zone, Z'			
			6	
			Region of DCR chart	
Condition	2, 3	4, 5	Region 1* or region 2	Region 3
Wall of single storey buildings	20	16	16	13
Multistorey buildings				
First storey walls	20	18	16	15
Top storey walls	20	14	14	9
All other walls	14	16	16	13
Parapets	4	2.5	1.5	1.5

*Only if qualifying crosswalls are present.

lead to the measurement of a hysteretic behaviour during earthquakes and, thus, of effective "ductilities." The mechanisms responsible for this hysteresis are not perfectly understood, but sliding friction on opened cracks and rigid-body rocking are known to be contributors. This is particularly true of the response of unreinforced masonry walls, which, under certain conditions, can remain stable much beyond cracking, as demonstrated analytically and experimentally. However, this stability is not infinite, and generates damage throughout its development. Although there exist approximate and simple hand-calculation models that capture the many ultimate failure modes reported in this paper, there is currently no broadly accepted integrated analytical model capable of replicating all possible modes of behaviour in a format compatible with modern inelastic nonlinear seismic analysis computer programs, nor is there agreement on what constitutes a reliable and acceptable value of "ductility" for unreinforced masonry elements. However, with the growing awareness that first cracking is not automatically equivalent to failure, more experimental and analytical research on the seismic performance of unreinforced masonry buildings is anticipated.

In the meantime, the new knowledge generated by some of the reported experimental programs has been directly incorporated into a standard seismic evaluation procedure in California, and has been used for the retrofitting of numerous unreinforced masonry buildings, even though some aspects of this procedure are largely empirically based and somewhat controversial. The legal protection provided by seismic hazard mitigation ordinances that endorse this special evaluation procedure has been instrumental in its broad acceptance there. This Californian special procedure has been recently "imported" and modified to be compatible with the Canadian context. Although its use remains voluntary here, and has not yet been endorsed by any legal document, it provides a credible alternative seismic evaluation method applicable to a certain type of unreinforced masonry buildings.

For the cases in which this approach is not applicable, dynamic analysis or finite element analyses can be contemplated. However, only scant information exists on the dynamic damping properties of these types of construction. Awaiting more definitive knowledge, conservative values, preferably in the lower range of reported values, could be chosen for analytical studies. Alternatively, forced-vibration analyses would be one way to obtain these data for individual structures as needed. As for finite element analyses, cracking models (the smeared-crack model in particular) are promising, although guidance for their application in a production environment is lacking. In the meantime, linear elastic finite element analyses have been of some use in spite of their obvious limitations.

So far, in Canada, to protect themselves against possible litigations following an earthquake, structural engineers have often adopted conservative analytical models in their seismic evaluations of unreinforced masonry structures; this has often translated into expensive rehabilitation costs. It is hoped that the concepts, underlying assumptions, and applicability of the new CGSEEB procedure, emphasized in this paper within a more comprehensive state-of-the-art review of this topic, will provide an added perspective on the problem, an overview of both existing solutions and those under development, and actual savings in future seismic rehabilitation costs.

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