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MASONRY DURABILITY

G.T. Suter[1]

ABSTRACT

The paper presents an overview of the key factors which must be controlled to achieve adequate masonry durability. These factors include material selection, design detailing appropriate to a project's environmental loading, workmanlike construction, and periodic maintenance. The paper makes a case for the proper durability design of masonry.

INTRODUCTION

Masonry durability may be defined as the ability of a masonry building/structure or any of its masonry components to perform its required functions over a period of time (author's interpretation from Ref. 1). Durability failure means loss of performance as defined by the onset of any of the following limit states (1):

(a) collapse, as it relates to human safety or to loss of function of the building (or other masonry construction);

(b) local damage, as it relates to loss of function of the building component or to appearance;

(c) displacement, as it relates to loss of function of the building component or to appearance;

[1]Professor, Department of Civil and Environmental Engineering, Carleton University, Ottawa, Ontario, Canada, K1S 5B6, and President, Suter Consultants Inc., Consulting Engineers, Nepean, Ontario, Canada, K2E 8A5.

(d) discoloration, as it relates to appearance of components having an aesthetic function.

While masonry has a longstanding and well-deserved reputation as a durable material, this reputation at times has been questioned particularly over the past twenty years as durability failures have occurred in building envelope systems. Dalgliesh (2) reported in 1992 that the building envelope is typically the system most prone to premature deterioration in a building. In 1989 alone, unsettled claims against Canadian architects and engineers for failing facades reached $50 million, and the Ontario New Home Warranty Program paid out $29 million for facades over a two-year period (1). While these claim and repair costs pertain to a range of cladding materials, based on my experience I would estimate that more than 50% are associated with masonry cladding systems. Masonry durability failures are not unique to Canada; I have been privileged to work on a number of challenging masonry cladding problems in the United States, and in my travels have observed a variety of masonry durability failures in other countries, be these countries in a hot or cold, dry or moist climate. All materials of course deteriorate with time and so it is to be expected that one encounters masonry durability failures. However what is of utmost importance to all of us involved in the world of masonry, is that masonry firstly, does not fail prematurely before it has reached its design service life and secondly, does not fail in a manner which endangers the safety of the public or causes great economic loss.

This paper makes a case for the proper *durability design* of masonry. You may ask: Is durability design not adequately taken care of in our present masonry design? The answer is no because firstly, it is hidden within our current design approaches and secondly, proper durability design today is still part art and part science. I use the word "hidden" here because the present day's limit states design approaches implicitly include durability considerations yet typically do not spell them out. Using the National Building Code of Canada (3) as an example, limit states means "those conditions of a building structure in which the building ceases to fulfill the function for which it was designed". The Code continues with an explanation that

> "those states concerning safety are called ultimate limit states and include exceeding the load carrying capacity, overturning, sliding, fracture and fatigue, while those states which restrict the intended use and occupancy of the building are called serviceability limit states, and include deflection, vibration, permanent deformation and cracking."

The word "durability" does not appear in any of these statements, yet aspects of a durable design would implicitly be achieved in following the ultimate and serviceability limit states design approaches. Does that situation suffice? I don't think so, because too many masonry durability problems occur in practice and as stated earlier, proper durability design today is still too much of an art than a science. To make it more of a science we need firstly, standards that explicitly address building durability, secondly, industry agreement about the expected service life of various types of masonry construction and components, and thirdly, widely accepted criteria of just what key environmental loadings and agents affect masonry durability over the longer term. Important developments to address these issues in North America are currently under way and these will be commented on as part of this paper.

STANDARDS

The Canadian Standards Association (CSA) is responsible for the production of masonry standards in Canada just as the American Society for Testing and Materials (ASTM) carries out this function in the United States. Masonry durability is affected by materials, design, construction, maintenance and environmental loads including deleterious agents. Currently, CSA standards exist to deal with materials, design, and construction and also partly with environmental loads. Key durability aspects of these standards are as follows:

Material Standards: Masonry Units

For the relatively harsh environmental conditions existing in Canada (as well as in the Northern United States and many other countries), freeze-thaw durability of masonry units is a major concern and CSA standards address this issue in some depth. The following discussion will be directed specifically at clay bricks because they are more susceptible to freeze-thaw durability problems than concrete and stone units.

CSA Standard A82.1-M87(4) outlines grade of brick requirements according to a Canadian weathering index, which for any locality is the product of the average number of freezing cycle days and the average annual winter rainfall. Three weathering regions of negligible, moderate and severe weathering have been defined as shown in Fig. 1. In essence, the Standard requires that a grade SW (severe weathering) brick be used throughout Canada for exterior masonry construction.

For SW brick, the Standard permits acceptance of brick if the average saturation coefficient is less than 0.78, the average compressive strength exceeds 20.7 MPa, and the average maximum 5-hour boiling water absorption is less than 17%. Note that previous editions of the Standard allowed a much more liberal maximum saturation coefficient of 0.88 and that the more stringent 0.78 value was adopted to reduce the frequency of freeze-thaw durability failures in the field; this also brought the value in alignment with similar ASTM standards in the United States where Grimm already in 1956 had developed a weathering index for bricks (5).

Unfortunately, the freeze-thaw resistance is not a definite property of a porous material as compared to, say, compressive strength. Whether or not a brick fails during freeze-thaw depends on the pore structure, the rate of freezing and the thickness of the material. The Standard recognizes this complex situation firstly, by allowing a standard 50-cycle freeze-thaw test to supplant the requirements of saturation coefficient and 5-hour boiling water absorption, and secondly, by providing valuable guidance to the designer in a non-mandatory appendix. In spite of the Standard's mandatory requirements and non-mandatory advice, freeze-thaw durability failures still occur too frequently in the field. In my opinion those failures are mainly due to three reasons. *Firstly*, the Standard's criteria of strength, absorption and saturation coefficient are rather unreliable predictors of freeze-thaw durability; they accept as many as 20% of bricks that will fail the freeze-thaw test and reject about 30% of the resistant ones (6); though the freeze-thaw test is regarded as a more reliable predictor of in-situ performance, it simply indicates if a brick wetted to a certain rate for an arbitrary number of cycles will fail or not. In-situ conditions may impose a much larger number of freeze-thaw cycles under differing climatic conditions which over the long term may be more severe than a standard freeze-thaw test. *Secondly*, freeze-thaw failures typically occur only if clay bricks are saturated to near the limit of their absorption capacities; this means that aspects of design, construction and maintenance must ensure that clay brick masonry is not subjected to excessive amounts of moisture. *Thirdly*, too few designers appear to heed the Standard's guidance in Appendix B where it is pointed out that due to all the present uncertainties of selecting a brick with adequate freeze-thaw durability, the user may be guided by the record of field performance of any particular brick. Based on my own experience, the performance record of a brick should be checked under varying exposure conditions for at least five years.

Aside from freeze-thaw durability failures, masonry units may experience an appearance durability failure due to efflorescence. For a rating scale of "no efflorescence", "slightly effloresced" and "effloresced", CSA standards typically permit masonry units rated no worse than "slightly effloresced" to be used in masonry projects. Since for efflorescence to occur, both water soluble salts and moisture must be present, and also since such salts may originate not only in the brick but in the mortar, backup, building trim made of precast concrete, natural stone or cast stone, and ground water, the determination of the root cause of efflorescence is always of vital importance. In practice it is at times very difficult to ascertain the source of moisture, whether from infiltration, exfiltration, a combination of the two or from direct moisture paths at cracks and architectural detailing. Again, aspects of design, construction and maintenance must ensure that masonry is not subjected to excessive amounts of moisture in order to control efflorescence.

Material Standards: Mortar

The new CSA mortar standard A179-94(7) addresses durability requirements in various sections of the document; in the following discussion, three key sections will be commented upon. *Firstly*, there is a new requirement that the commonly used Types S and N mortar shall produce a minimum flexural bond strength of 0.20 MPa in masonry construction. This lower limit of strength will help achieve better durability because it requires some proven level of compatibility between mortar and units; this in turn will help control masonry cracking and moisture uptake; finally, lower moisture levels in the masonry will help control durability failures such as freeze-thaw deterioration and efflorescence. *Secondly*, the Standard sets out limits on the water-soluble chloride ion content in mortar and grout. This new requirement will help control premature corrosion of embedded steel such as joint reinforcement, connectors and reinforcing bars. *Thirdly*, the Standard's non-mandatory Appendix A presents valuable guidance on the selection of mortar types not just for compressive strength but for bond strength and durability. I fully agree with its recommendation that for a particular project one should select "the weakest mortar in compression that is consistent with the performance requirements of the project". Guidance, however, is not provided on the minimum air content of mortar for adequate freeze-thaw durability. While the Standard sets an upper limit of 18%, no lower limit is specified nor discussed in Appendix A. Since Davison (8) has shown that the durability of Portland cement-lime mortars containing 10-15% air is superior to those containing 4 to 7% air, a lower limit of about 10% is advisable for mortars subjected to severe freeze-thaw conditions. Masonry cement mortars typically contain 10 to 15% air and therefore require no additional air entrainment for adequate durability (9); Portland cement-lime mortars on the other hand would require the addition of an air entrainment agent to reach a 10% level (9).

Material Standards: Connectors

Connectors, that is ties, anchors and fasteners, are a vital part of modern masonry construction and therefore a major contributor to masonry durability. To define critical aspects of masonry connectors such as materials to be used, corrosion protection, strength/stiffness criteria, fabrication and shape, a CSA Connector Committee was set up in the 1970's. It's chairman, Bruce Hastings, reported on the committee's draft standard at the Second Canadian Masonry Symposium in 1980 (10); issued in 1984 as CSA Standard A370(11), this document was the first standard in North America to deal exclusively with masonry connectors. Since its development was partly driven by firstly, the extensive use of thin veneer walls attached to more or less flexible backups and secondly, the incidence of major cladding failures, its introduction has had a significant effect on Canadian practice and connector research and development work. The latter work together with an industry-wide concern about the long term corrosion resistance of connectors recently led to the introduction of the second edition of the Standard, A370-94(12). From a masonry durability point of view, there are two major mandatory changes in this edition: Firstly, there are improved clauses on corrosion in which three levels of corrosion protection are matched with nine classes of exposure environment. Secondly, stainless steel or equivalent durable materials are required for ties in buildings over 11m high which are located in areas where the annual driving rain index is either "moderate" or "severe". As can be seen from the annual driving rain index map of Canada in Fig. 2, both the Atlantic and Pacific coast regions as well as the Great Lakes regions in heavily populated Southern Ontario come under this new and stricter mandate. The Standard additionally provides valuable guidance to designers on the corrosion of metal connectors in the non-mandatory Appendix C. Besides comments on the life of connectors ("they should have sufficient corrosion protection to enable them to function effectively for the expected life of the building. The lifespan of buildings varies according to their individual functions and requirements, but 50 years should be considered a minimum goal for the design of most institutional and high-rise buildings.

The life of connectors can be difficult to predict since many factors influence corrosion. There is little documentary information on the performance of connectors built into walls under Canadian conditions. Much therefore depends on judgement and experience."), information is given on factors affecting corrosion, connector composition, environmental factors (such as annual average levels of sulphur dioxide in selected cities and the acidity of the rain in eastern Canada), detailing and workmanship, and contact between different metals. All of this guidance is of vital importance for the long term durability of masonry since firstly, the corrosion of metals is a complex issue, secondly, environmental loads and deleterious agents are difficult to quantify, and thirdly, the effects of premature corrosion can be disastrous in terms of endangering the public and cost of repairs.

Design Standards

Masonry design for Canadian buildings is carried out according to CSA standards produced by the S304 committee. The committee later on this year will issue a new limit states based standard which for the first time will contain a specific section dealing with durability requirements (13). That section states very simply "masonry and its components shall be designed for durability" and then provides a number of notes which, although non-mandatory, are of great value. From the notes reproduced in Table 1 it can be seen that firstly, special durability design considerations must be given to structures exposed to unusual environmental conditions and secondly, the buildup of excessive moisture in the masonry must be prevented to reduce the risk of durability failures such as freeze-thaw damage and premature metal corrosion. The section's new durability requirements and guidance should go a long way towards achieving durable masonry designs in the future.

Another soon to be issued CSA document will have an impact on the durability of Canadian buildings in general and therefore also on masonry structures and masonry components. That document is CSA S478 "Guideline on Durability in Buildings" (1). As quoted from its preface, "this Guideline sets forth for the first time in North America a set of recommendations which will assist designers in creating durable buildings. A framework within which *durability targets* may be set is provided and criteria for defining *durability performance* of buildings in terms commonly used, but previously unspecified, are suggested.....It contains generic advice on the environmental and other design factors that impact on the durability of building components and materials, and identifies the need to adjust material selection decisions by consideration of initial and long term costs, maintenance possibilities, replaceability, and importance of major and minor components".

"The Guideline makes it clear that design choices which may impact on durability and service life requirements should be thoroughly discussed and agreed upon among all concerned, in particular the owner, designer, and constructor, and provides a model document for recording these decisions in Appendix A. Later Appendices discuss and expand upon issues related to identification and (relative) quantification of environmental loading, deterioration mechanisms, and damage avoidance strategies including the need for appropriate maintenance over the life of the building."

Key recommendations from this Guideline which pertain to masonry durability are the following:

1. Regular maintenance should be incorporated in the *design service life* of a building in terms of such issues as frequency, nature, feasibility and cost. Note that design for maintenance is not included in any other design standards.

2. A statement of the design service life (in years) for both the building and its components and assemblies should be provided and accepted by the owner as the agreed basis for the design. Typical design service lives, as interpreted from the Guideline for masonry construction, would be 25 to 44 years (medium life) for most industrial buildings, 50 to 99 years (long life) for most residential, commercial, institutional and office buildings, and greater than 100 years (permanent) for heritage and monumental buildings.

3. Durability design considerations should include material selection, detailing, buildability, and operation and maintenance. Of particular masonry relevance are detailing requirements to minimize moisture buildup and buildability requirements that should incorporate the input of contractors, fabricators and suppliers. The Guideline further recommends that the design should allow for ease of access for inspections, testing, maintenance and repair throughout the building's design service life.

4. Timely repairs are judged to be of great importance. It is recommended to carry out visual inspections as part of regular maintenance and to promptly rectify deterioration problems.

In addition to these key recommendations, the Guideline presents especially useful information for the masonry designer in two appendices. Appendix D provides an overview of building envelope durability issues pertaining to: roof and associated structures; exterior walls including cladding and windows; below-grade walls and floors; cantilevered floors and soffits; and connecting joints. Appendix E presents a comprehensive summary table of deterioration mechanisms for building materials and their control. Of particular interest to the masonry designer is the information on stone, clay brick, concrete block, mortar and metals. The tabular information can serve as a checklist to ensure the use of durable materials for a durable masonry design.

Construction Standard

The recently updated CSA A371-94 Standard (14) governs the masonry construction of buildings in Canada except for housing and small buildings which are built according to the Part 9 requirements of the National Building Code of Canada (3). To achieve durable masonry construction, the Standard includes all the basic requirements for controlling moisture (such as provision of weeps, vents, air space, "reasonably clear" cavity, flashing, damp-proofing, filling of joints, and preferred joint configurations) and premature corrosion of embedded steel (such as minimum cover requirements of ties and reinforcing steel). For the commonly used cavity and veneer wall systems, the requirements for a minimum air space and cavity (see Table 2) for the first time clearly recognize construction tolerances and construction practices. Although some of the requirements are contained in non-mandatory notes, the guidance provided is valuable and should be heeded by both the designer and contractor. Note that the minimum air space of 25 mm is to be widened to at least 40 mm "whenever the additional air space is relied upon to provide resistance to the ingress of precipitation". Even if only the 25 mm width is provided, with a permissible variation of $\pm$13 mm, continuity of the air space should be ensured. The requirement that the cavity be kept "reasonably clear of mortar fins and droppings", rather than "free of mortar" as in previous editions of the Standard, recognizes normal workmanlike construction practices.

PUBLICATIONS

Masonry durability undoubtably has been a concern for the thousands of years that masonry has been mankind's foremost construction material and there are hundreds of publications that deal with this topic. In the following I wish to comment only on a few selected publications that are broadly based or else introduce a Canadian component.

1. Grimm (15) provides an excellent summary of the publications since 1900 dealing with the durability of brick masonry. His review is structured according to destructive agents, mechanics of destruction, porosity, freeze-thaw resistance, mortar properties, florescence, environment, architectural engineering, brick specifications, construction, and maintenance. A total of 228 publications are cited. He concludes with these words: "Good materials and design will not result in durable brick masonry without good construction workmanship and proper maintenance. The manufacturers of brick, lime, cement, and sand, the architect and engineer, the contractor, and the building owner must share responsibility for the durability of brick masonry. Keeping the masonry as dry as possible is the single most important variable in masonry durability."

   The paper's contents concerning durable brick masonry can be extended to serve as the basis for achieving durability of concrete and stone masonry as well.

2. Hutcheon and Handegord (16) in 1983 produced a book on the building science for a cold climate which has been recognized as a landmark publication. While the 440-page book deals with building science topics applicable to all forms of building construction, there are many topics involving buildup, penetration and deposition of moisture which directly impinge upon masonry durability.

3. Brand's 1990 book (17) contains architectural details for building envelopes which include brick and stone cladding as well as other cladding materials. Although no

treatment is given to the widely used brick veneer/steel stud wall system, there are other valuable details and discussions to affect moisture control and hence achieve better masonry durability.

4. Drysdale's and Suter's 1991 book (18) on cavity and veneer wall systems (including the brick veneer/steel stud system) provides guidance to designers and contractors on how to achieve masonry wall system durability. Specific chapters deal with durability related issues such as movement joints, building science requirements, parapet and roof connections, masonry efflorescence and freeze-thaw resistance, surface coatings, maintenance and repair. The authors recommend that every design be subjected to a *vulnerability assessment* before a final decision is made. The vulnerability assessment should include:

   - extent to which design objectives (requirements) are satisfied;
   - cost and occupant acceptance;
   - adaptability to building tolerances;
   - sensitivity to construction conditions (e.g. temperature, humidity, cleanliness);
   - integration with other components (e.g. windows, interacting walls);
   - construction sequence and interaction with other trades;
   - constructability (e.g. simplicity and repeatability of details and minimization of components, dissimilar materials, and different trades);
   - inspectability and ability to identify and correct faults during construction;
   - long term durability and life of components in the environmental conditions of the project; and
   - ability to provide maintenance and to repair faults.

   While only the last two items directly refer to masonry system durability, all factors of the vulnerability assessment indirectly contribute to long term satisfactory performance.

5. Gazzola (19) at the last Canadian Masonry Symposium provided a valuable critical review of all Canadian masonry material standards including clay masonry units, concrete masonry units, stone masonry units, mortar and grout, and connectors/reinforcement. He concluded that "the majority of the Canadian masonry material standards require further improvement with regard to durability provisions. The reason for their inadequacy is generally considered to be as a result of unrepresentative durability test techniques or the lack of or incomplete coverage of all of the various parameters which influence durability."

6. Maurenbrecher and Brousseau (20) in 1993 presented a state-of-the-art review of the corrosion resistance of metal components in masonry cladding. After commenting on the then current code requirements in Canada, USA and many European countries, as well as discussing factors affecting corrosion in masonry

cladding, they conclude that "until further studies show otherwise, ties with greater corrosion resistance than specified in current North American standards should be considered for highrise buildings, buildings with a known long life, buildings adjacent to heavy industry and buildings in areas directly exposed to marine climates, or severe or moderate driving rain." Their work and recommendations are now reflected in the more stringent durability requirements of the 1994 connector standard dealt with in this paper's last section.

KEY MASONRY DURABILITY FACTORS

General

The review of CSA standards and publications indicates that to achieve satisfactory design service lives for masonry structures and masonry components, close control must be exercised over materials, design (and especially detailing), construction, and maintenance. The control over these four basic factors must be appropriate for the environmental loading of a particular project. In dealing with masonry durability problems on many projects, it has been my experience that a major problem is rarely due to deficiencies or lack of control in just one of the four factors of materials, design, construction and maintenance; instead, problems typically can be traced to deficiencies in two, three or even all four factors. The complexity of masonry durability failures then makes it all the more important that a proper *durability design* become a standard feature of overall design and that such a design include input from all parties involved in a project: designers (architect and engineer), material manufacturers, suppliers, contractor(s), and owner.

Grimm (15) concluded his review of durability of brick masonry with the statement that "keeping the masonry as dry as possible is the single most important variable in masonry durability". I agree with that statement but would add that to prevent most durability failures, the control of the three M's is vital: <u>m</u>ovements, <u>m</u>oisture, and <u>m</u>aintenance. Again, a complex interaction between these three durability factors or variables often occurs; for example, if movements are not properly accommodated, cracking takes place which permits moisture penetration and requires maintenance. The following sections will deal with selected three M requirements under Canadian climatic conditions.

Movements

Due to a variety of movement restraint conditions (18,21), high and long masonry walls are prone to cracking. To minimize such cracking, movement joints are typically provided in the walls.

For cavity and veneer walls in highrise structures, Fig. 3 illustrates that differential movements between a continuous brick masonry veneer and a reinforced concrete frame would be very large (18). It has been recognized for over 20 years that if such differential movements are not properly accommodated by *horizontal* joints typically at one-to two-storey intervals, durability failures such as brick spalling, mortar crushing, and veneer panel buckling may take place as shown in Fig. 4 (22). For the common case of veneer supported on shelf angles, Fig. 5 provides a detail with a clear vertical movement gap which will prevent such durability failures (18). Fig. 6 illustrates the case of a large brick masonry chimney enclosure which had different support conditions for the front half compared to the back half and as a result developed cracking problems towards the chimney top. Each vertical movement case therefore must be carefully thought through and detailed by the designer.

Long horizontal walls typically require *vertical* movement joints to control cracking. While for instance BIA's recommendation (23) that joints be placed in long walls and at offsets, junctions, corners, and parapets represents valuable guidance, the required frequency of joints remains part art and part science. This is evident from the CIRIA field study carried out in England on long masonry walls built of both brick and concrete masonry (24). A total of 85 buildings consisting of either framed structures with masonry cladding or loadbearing masonry were examined for evidence and nature of cracking. Table 3 provides the number of instances of cracking or movement recorded according to eight categories. In Figs. 7 and 8 the number of cases of "no" and "slight" cracking have been combined and contrasted with those of "moderate" cracking; these have been expressed as a function of the unbroken length of the wall (defined as the length between corners of a building, or adjacent full height movement joints, or other discontinuities such as door openings). Note that the BS 5628 limits refer to the recommended movement joint spacings of 12 m for brick masonry and 6m for concrete masonry in the British Standards Institution's BS 5628 guidelines. The results of Table 3 and Figs. 7 and 8 indicate the following:

1. More than half the buildings exhibited signs of movement and distress.

2. Walls shorter than the 12 m and 6 m limits exhibited much less distress than longer walls.

3. Walls of intermediate length (say 2 to 3 times the 12 m and 6 m limits) were more prone to crack.

4. Surprisingly, very long walls appeared to exhibit less distress than intermediate length walls.

These findings led to the CIRIA conclusion that besides length, certain wall configurations affect cracking from in-plane longitudinal movements (24). Their listing of configurations more or less vulnerable to cracking is given in Table 4.

All of these considerations reinforce the earlier statement that the provision of vertical movement joints to accommodate horizontal movements is still part art and part science. Since such movements are also very much influenced by temperature, designers must heavily rely on their judgement and experience in a particular climatic region to arrive at joint locations that will prevent a durability failure.

The effect of a movement related durability failure can be minor to major. It would be minor if for instance hairline cracks in joints due to concrete block shrinkage were acceptable from an appearance point of view or else would require only minor repointing; it would be major if the absence of the movement joint shown in Fig. 5 caused the sudden buckling failure of storey high veneer panels endangering the public and requiring major repairs. Whatever the range of effects, cracked masonry may permit the ingress of excessive amounts of moisture which in turn accelerates other moisture induced distress such as corrosion of embedded steel. Movement durability failures can be attributed to aspects of materials, design, construction and maintenance; the main culprit in my experience typically has been design.

Moisture

The major masonry related durability failures of corrosion of embedded steel, freeze-thaw deterioration of masonry units and mortar, efflorescence and other aesthetic distress such as staining, as well as mortar erosion are all moisture driven. Just how pervasive the effect of moisture is in contributing to durability failures can be seen from Fig. 9 which illustrates moisture's involvement in the deterioration of a single masonry component, namely mortar. While this source-mechanism-effect relationship (25) shows the complex interaction of many deterioration sources and mechanisms, moisture is involved as the source problem in every one of the final effects, be they efflorescence, cracking, spalling, disintegration, etching, erosion, or staining.

Moisture can enter masonry through what can be termed direct paths and in the case of masonry wall systems, through infiltration and exfiltration; in the case of exfiltration, under cold climatic conditions warm humid air from the interior can escape through defects in the continuous air and vapour barriers, and condense as moisture in the wall system. Whatever the source of moisture, whether it arrives in the masonry or masonry wall system directly or indirectly, the most common moisture related durability problems in my experience have been freeze-thaw deterioration of clay masonry units, efflorescence and connector corrosion. The following masonry elements and system components are particularly prone to fail due to moisture effects and therefore require special care in the selection of materials, design, construction and maintenance:

1. *Sills, copings and paving:* these masonry elements are directly exposed to moisture from rain, snow and ice, and to survive must act somewhat like a roofing material which is an impossible long term task. All masonry is absorptive, partly through the units themselves, partly through the mortar, and above all through even small cracks or incompletely filled mortar joints. The field conditions then promote high saturation levels in the masonry and upon repeated freezing and thawing can cause frost deterioration especially in the case of clay units or overall disintegration of masonry as a whole.

In my experience, if clay brick sills and copings are to be used under the cold weather conditions typical of most of Canada, only bricks with a proven, satisfactory 10-year minimum performance record should be selected. Even then detailing must ensure the effective shedding of moisture through adequate slopes and drips; furthermore, construction must ensure above-average workmanship in such matters as filling and tooling of joints; also, periodic maintenance requirements must be carried out promptly. I do not recommend clay brick paving for exterior Canadian applications in the regions east of the Rocky Mountains with which I am familiar; on the other hand, concrete brick paving in my opinion is typically durable especially when laid without mortar joints. Aside from potential frost deterioration, sills, copings and paving may exhibit efflorescence if sufficient water soluble salts are present.

2. *Walls below solariums:* in the last 10 to 20 years an increasing number of designs have featured solariums and other large sloping glass surfaces. When such building features tie in with masonry walls below them, the masonry receives the moisture runoff and again must act somewhat like a roofing material. The resultant buildup of moisture in the masonry may cause similar problems as discussed for sills, copings and paving. Additionally, the frequent flow of water may cause firstly, mortar erosion, secondly, staining of the masonry, and thirdly, premature corrosion of any embedded steel such as ties. To prevent these problems, special detailing is required to channel the water away from masonry walls.

3. *Walls below scuppers:* scuppers gathering water from large surfaces such as roofs and balcony slabs at times penetrate masonry walls as at balcony upstands or building entrances. Due to poor scupper detailing, scupper maintenance or partial ice/snow blockage, runoff from scuppers can be directed over the face of masonry walls. The resultant high moisture conditions in the masonry can be similar to the solarium situation, hence analogous durability problems may occur.

4. *Walls affected by splashback and landscaping:* the absence or inadequate maintenance of eavestroughing can create moisture saturation of masonry walls at grade level due to splashback of rainwater; additionally, poorly sloped landscaping and improperly located downspouts can also direct moisture into masonry at grade level. These high moisture conditions typically at the bottom of a wall just above the foundation level can cause efflorescence, mortar deterioration, corrosion of embedded steel, corrosion especially of bottom tracks of a steel stud backup system, and frost deterioration in the case of clay bricks. A minimum vertical separation of 200 mm between clay brick masonry and grade level in my experience is required to ensure clay brick frost durability for those conditions.

5. *Walls affected by other excessive runoff conditions:* especially historic stone masonry structures such as church towers and monumental buildings incur moisture damage from roofs and other details channelling water unto masonry elements such as walls, piers and foundations. Two types of durability failures are common: firstly, the soft lime mortars erode away, allowing moisture to penetrate deeply into massive wall sections and over the years dissolving the binder until little more than sand remains of the mortar; secondly, absorptive stones become highly saturated and under the action of frost and pollutants fail by exfoliation. A combination of regular maintenance and possibly re-channelling the water are appropriate measures. Because of the heritage nature of many of these structures, measures which can be taken to redirect runoff are often very limited.

6. *Parapets, chimneys, and wing walls:* all these masonry elements ("wing walls" are to denote walls that do not connect to an interior space) are exposed to more severe climatic conditions (above all, driving rain and frost action) than normal masonry walls. This means they are more susceptible firstly, to frost deterioration if clay bricks are used and secondly, to efflorescence. Cap details and flashing connections with the roof are critical to control direct moisture penetration and hence frost damage and efflorescence. Parapets especially on older buildings often exhibit cracking which permits additional moisture penetration. Parapet cracking is caused by the absence of vertical movement joints to accommodate horizontal in-plane movements mainly due to temperature. During high temperature periods, the walls expand and crack near parapet corners; since the cracked parapet cannot contract to its original length upon a decrease in temperature yet will expand further during the next high temperature period, the resultant ratcheting effect together with inappropriate repointing measures cause crack widening and an eventual leaning outward of at least portions of the parapet. Again, timely maintenance is required to minimize moisture penetration and associated wall distress.

7. *Planters and landscaping walls:* these elements act in a similar fashion to parapets except that the raised soil level on one side can create additional durability problems. For planters and landscaping walls to be durable, they must represent a bathtub design with many drains; this requires virtual perfection in the installation of waterproofing membranes not just around the interior perimeter of the element but also at each of the drains. Such perfection is not realistic and hence moisture penetration of masonry typically results not just from the top coping surface and the exterior masonry face but also from the soil side. Typical moisture related durability failures are frost deterioration of units (if clay brick) and mortar, efflorescence and corrosion of embedded steel. The problems at times are exacerbated by cracking from thermal movements and frost expansion of soils. If these elements are to be used in the harsh Canadian climatic conditions, frequent maintenance and failure must be expected.

8. *Soffits:* masonry soffits are really the antithesis of the age-old and time-proven compressive use of masonry. Soffits are tensile elements which depend on careful detailing and often fussy construction involving metal connectors and reinforcing. Even if a very durable austenitic stainless steel is used for connectors and reinforcing, somewhere further up in the construction such steel must typically be tied to more substantial steel members which will not be stainless steel. In the presence of moisture over prolonged periods of time, that connection may prove to be the weak link in the soffit assembly. In my experience, soffits constructed in the past with galvanized steel connectors and reinforcing have had relatively short service lives at times; it remains to be seen how durable such elements prove to be when stainless steel is used. A critical detail for soffits is the provision of a continuous effective drip at its edge to control the spread of moisture along the soffit's surface. I have never come across a freeze-thaw deterioration problem of a soffit but efflorescence and corrosion staining can occur.

9. *Veneered arches:* many veneered arches today do not act in compression and do not possess strong abutments to resist outward thrusts; instead, if designed properly at all, they are held up by steel straps and reinforcing in a somewhat similar fashion to soffits. Given slight movements due to temperature or shrinkage, such arches can crack and while remaining structurally safe due to the presence of the steel, this will allow increased amounts of moisture to enter the masonry. The same long term durability problems as discussed for soffits may occur.

10. *Glass masonry:* glass masonry exterior wall construction is susceptible to moisture penetration as are all single wythe masonry walls. The moisture can promote corrosion of embedded galvanized steel straps and reinforcement which in turn produces unsightly rust staining quite aside from eventual structural problems. It is recommended to use stainless steel for such applications.

11. *Ties:* as discussed earlier in the paper, ties used in masonry claddings in North America require increased corrosion protection to prevent premature corrosion failures. The extent of corrosion problems in the cladding systems of existing buildings in Canada has never been estimated but from limited investigations is believed to be significant. In the late 1980's, Suter Keller Inc. was commissioned by Canada Mortgage and Housing Corporation to examine eight highrise buildings employing brick veneer/steel stud wall systems for evidence of corrosion (26). The buildings were chosen in different climatic regions representing severe (St. John's, Newfoundland), moderate (Montreal and Toronto) and sheltered (Calgary) locations according to the driving rain index of Fig. 2. Corrosion of the galvanized ties was reported for six of the eight buildings. The buildings were 4 to 10 years old at the time of the study. What is of particular interest, is that the two buildings without observed tie corrosion were located in St. John's and Calgary thus representing severe and sheltered regions. The study clearly

indicated that other factors of material selection, design, construction and maintenance had a major impact on tie performance.

The new, more stringent CSA A370 connector corrosion requirements (12) should go a long way in overcoming premature tie corrosion in the future. Given that veneer walls are prone to at least some moisture penetration and also that ties (which are difficult to inspect and costly to replace) should not form the weak link in the veneer wall system, the prudent designer may well specify stainless steel ties on projects where the new standard would permit hot-dip galvanized ties.

In the past, ties and steel stud backups often have been too flexible and this has contributed to veneer cracking and resultant moisture penetration. Guidance on preventing these weaknesses has been published elsewhere (18) and the new CSA standards on masonry connectors and design (12, 13) now adequately address the stiffness requirements.

The consequences of moisture related durability failures can be minor to major depending on the type and extent of failure. The premature corrosion of ties at times has necessitated the costly installation of retrofit ties, and even the dismantling and reconstruction of a building's veneer.

Maintenance

Masonry constructions, as do all building materials, deteriorate to some degree with time because of weathering, aging, and other environmental effects. This deterioration process, which reflects various types of durability failures, should be controlled by timely maintenance measures. It is important to incorporate maintenance in the proper *durability design* of a project so that especially the owner is cognizant of planned future project costs. It is also important to recognize that it is cost effective to carry out planned-for periodic maintenance and minor repairs rather than allowing the deterioration process to proceed until major repairs or replacement are required. Figs. 10(a) and (b) illustrate this point schematically for a masonry cladding system (18).

To paraphrase from Ref. 18: As shown in Fig. 10(a), the soundness of a cladding system reduces with time, first slowly but later at an increasing rate. Assuming point B on the curve represents a degree of soundness which divides serviceability problems from safety problems (note that a serviceability problem such as cracking also represents a durability failure), then it is seen that periodic maintenance/minor repair actions are required at a time before t_B. If such actions are planned for and taken at a time t_A corresponding to point A, then system soundness is again similar to its original condition. The deterioration cycle would then repeat itself until at some future date, t_C, further planned-for maintenance measures are taken. Fig. 10(b) indicates that maintenance/repair costs escalate significantly as the level of soundness of the cladding system decreases. The schematic curves show that major costs arise once safety problems come into play and also that it is of economic advantage to commit maintenance/minor repair funds periodically rather than face major costs once the system has badly deteriorated.

It is clear that durability failures should never reach the deterioration stage where safety is compromised; this means that planned-for maintenance as part of durability design should be carried out at a time like t_A safely ahead of t_B. To define the precise scope and nature of periodic maintenance for a particular masonry project, inspections by a qualified professional are recommended at 2- to 5-year intervals depending on past experience in the climatic region of the country and the past performance record of the project.

CONCLUSIONS

The new and existing CSA standards together with the soon to be introduced CSA Guideline on Durability in Buildings provide the requirements of masonry materials, design, construction, and maintenance to achieve masonry durability for Canadian environmental loads. The designer should carry out a durability design check to ensure all aspects of durability have been considered. This design check requires input from material manufacturers, suppliers, contractors, and the owner. The vast majority of durability failures can be prevented by taking measures to control movements and moisture, and by carrying out timely maintenance.

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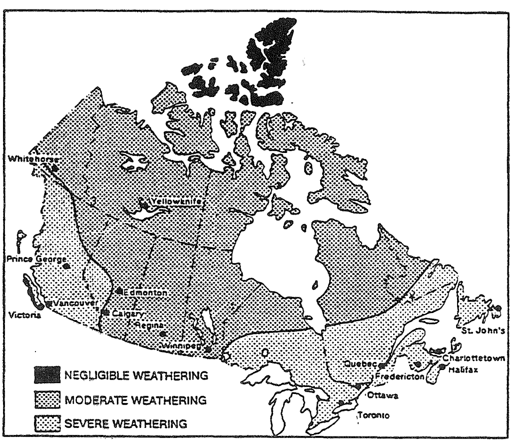


Fig. 1 Weathering index map of Canada

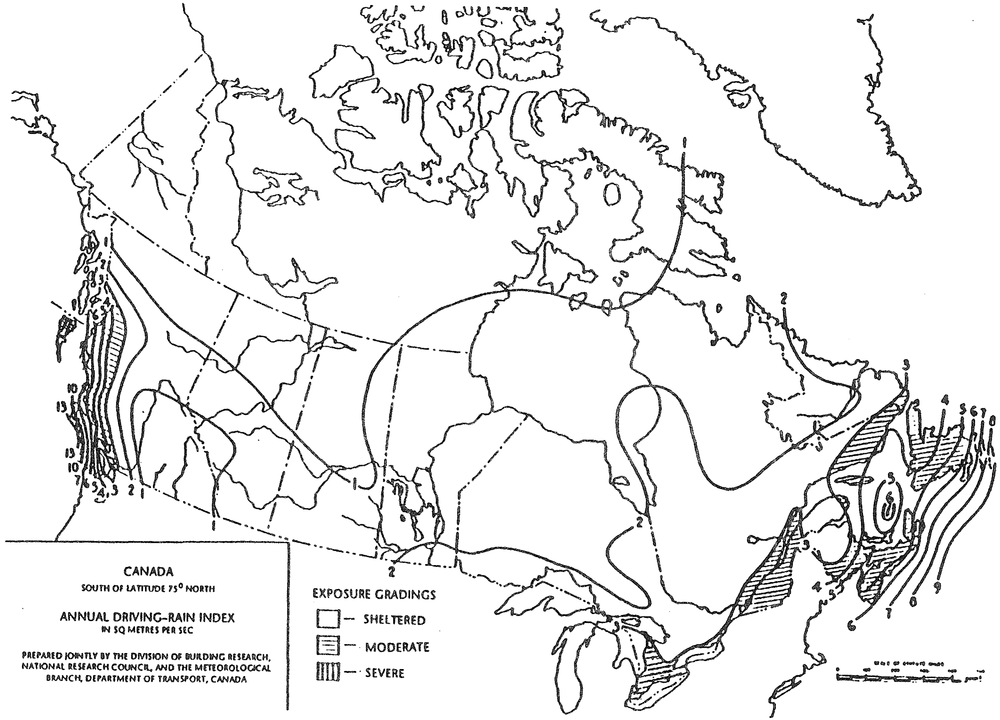


Fig. 2 Annual driving-rain index map of Canada

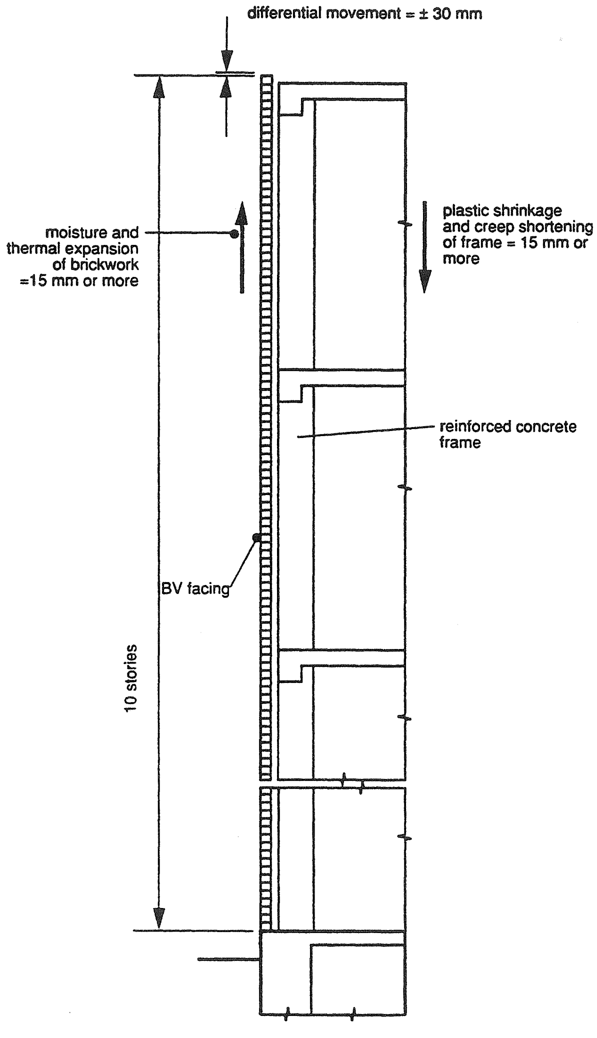
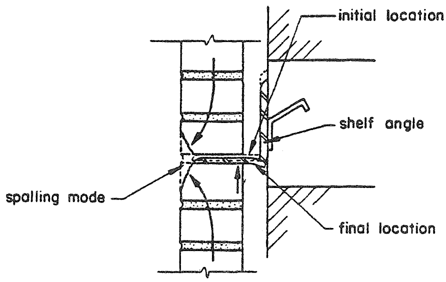
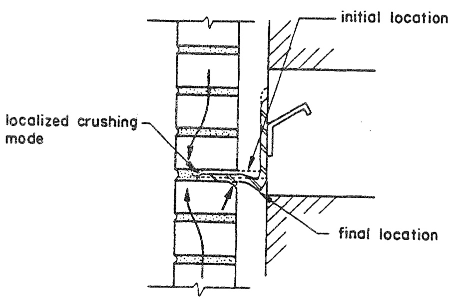


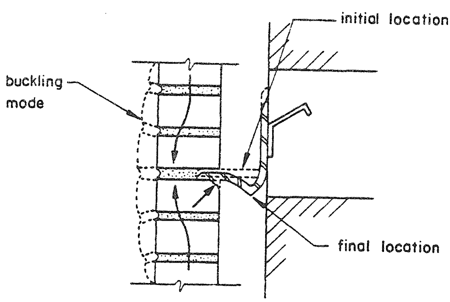
Fig. 3 Difference in height of clay brick veneer and frame if relative movements unrestrained



(a) Case of brick spalling for small overhangs

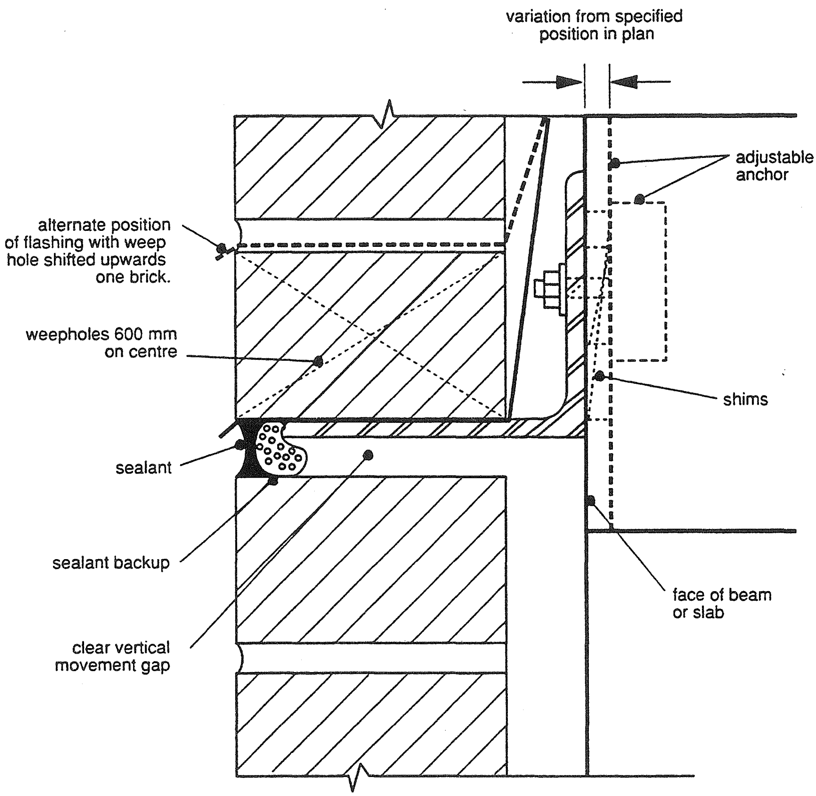


(b) Case of mortar crushing for medium overhangs



(c) Case of veneer panel buckling for large overhangs

Fig. 4 Potential veneer failure modes in the absence of horizontal movement joints to accommodate differential vertical movements (22).



Note: While some designers specify "soft" filler strips in the movement gap, such practice is not recommended for two reasons: some filler strips can transfer significant compressive loads from structure to veneer and an open gap can be more readily inspected.

Fig. 5 Detail of horizontal movement joint at shelf angle

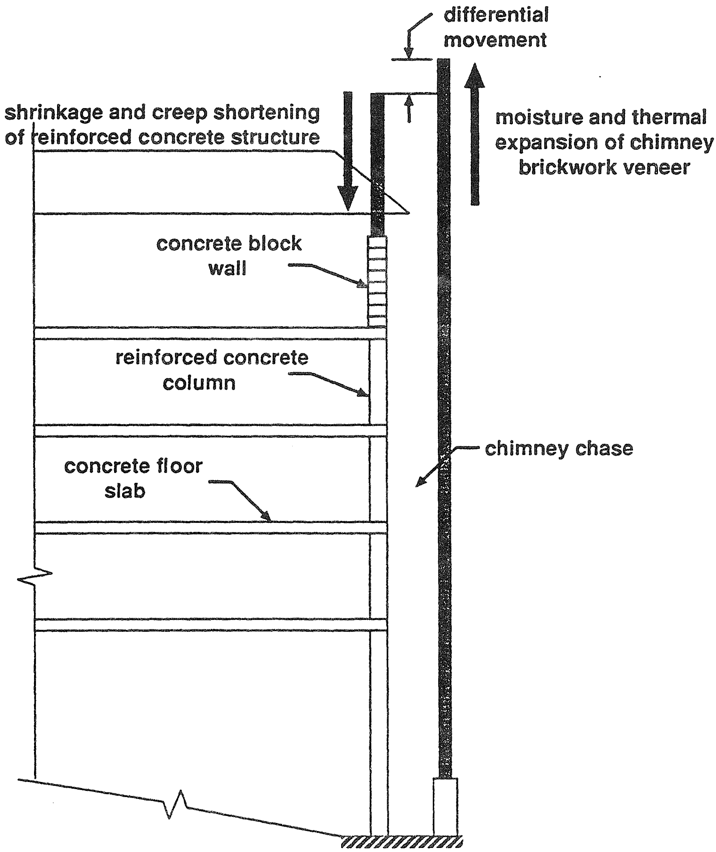


Fig. 6 **Schematic view of different veneer heights and support conditions of front and back parts of chimney indicating need for a movement joint. Cracking distress occurred towards top of chimney.**

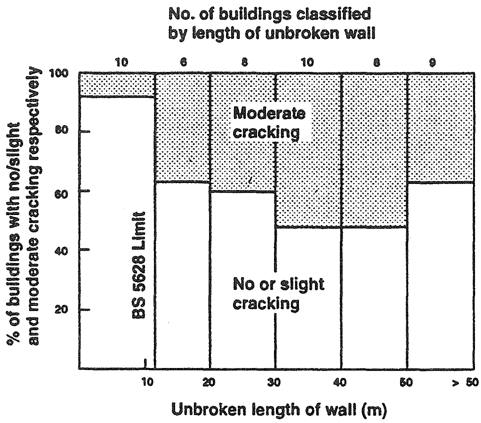


Fig. 7 CIRIA study's incidence of cracking in brick masonry (24)

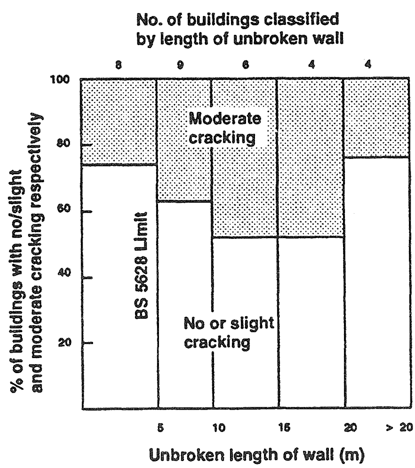


Fig. 8 CIRIA study's incidence of cracking in concrete masonry (24)

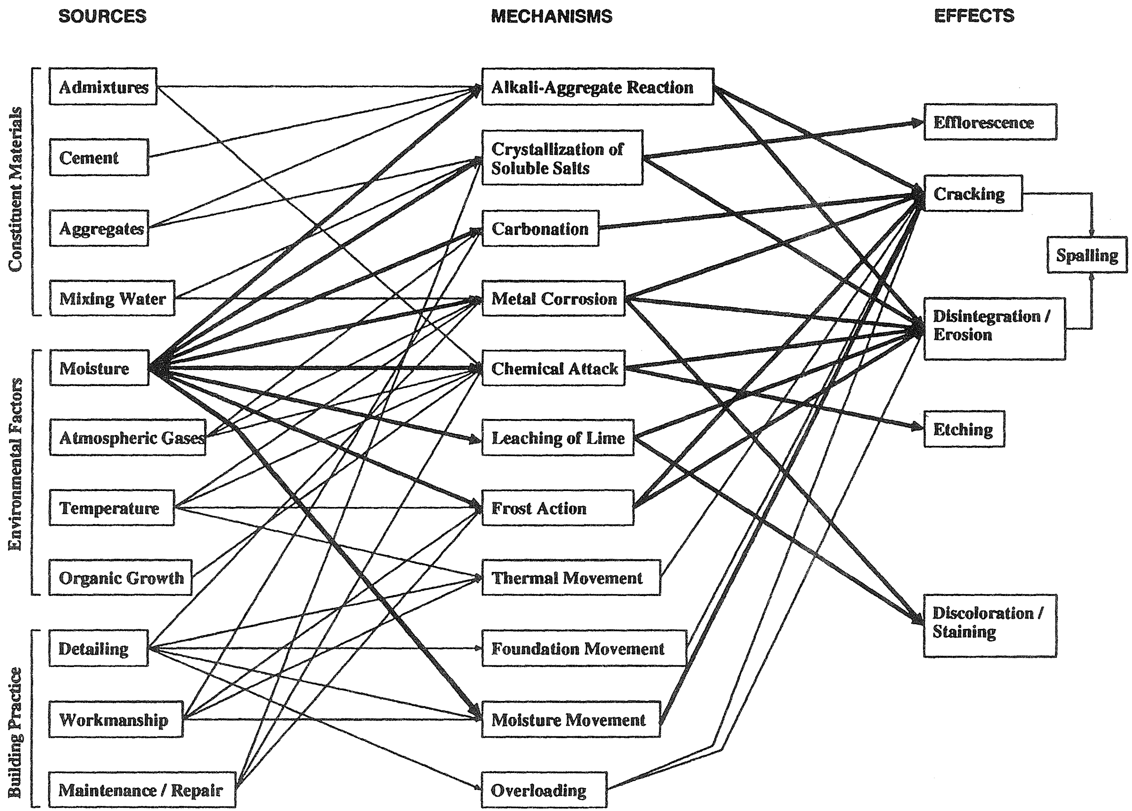
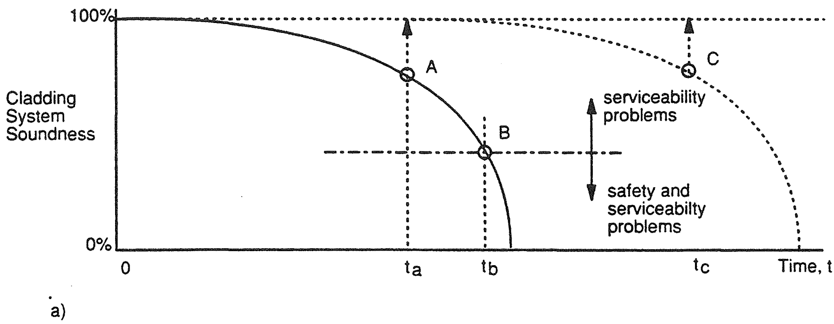


Fig. 9 Source-mechanism-effect relationship of mortar deterioration. Note that the heavier arrows connect moisture as the major deterioration source to all effects or durability failures.



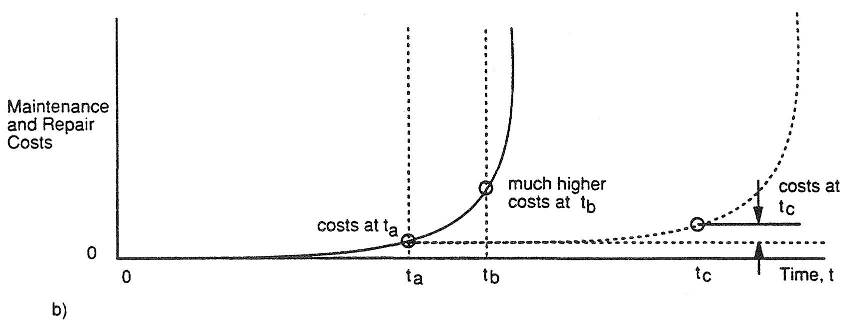


Fig. 10 **Schematic illustration of soundness and maintenance/repair costs versus time for a cladding system (18).**

Table 1 Excerpts from S304.1 for Durability Design
6.5 Durability **6.5.1 General** Masonry and its component shall be designed for durability. **Notes:** (1) Masonry material should be selected and masonry elements should be designed (a) to satisfy their function; (b) to achieve their performance requirements; and (c) to resist the loads, acting alone or in combination, to which they will be subjected within their anticipated service environment for their design service life. (2) Masonry structures designed in accordance with this Standard and constructed in accordance with CSA Standard A371 are considered to have met the requirements of Note (1). (3) For special structures, or structures exposed to unusual environmental conditions, the application of additional durability measures may be warranted. (4) A major factor influencing the durability of masonry is its moisture content. For example, excessive moisture may lead to freeze-thaw damage and accelerated corrosion of metal components. The use of durable materials and design details which reduce water ingress in the wall will reduce the likelihood of damage.

Table 2 Excerpts from A371 for Air Space and Cavity
5.13.3 Minimum Air Space **5.13.3.1** The design width of the included air space in cavity walls and veneer walls, and the permissible variation in the constructed width of this air space, shall be as specified by the designer. **Notes:** (1) Because construction tolerances for the masonry and the structural backing are normally accommodated by the air space, the width of the constructed air space will likely vary from the design width. (2) For unit masonry construction, where the control of precipitation into wall components, assemblies, or interior space is a requirement of the design, cavity walls and veneer walls should include an air space having a design width of (a) 25 mm, where the air space is relied upon to provide resistance to the ingress of precipitation; or (b) not less than 40 mm whenever the additional air space is relied upon to provide resistance to the ingress of precipitation. **5.13.3.2** Where the width of the constructed air space does not satisfy the specified permissible width variations, the designer shall be notified before the affected masonry work progresses. Unless otherwise specified, the permissible variation in the width of the constructed air space shall be ± 13 mm. **5.13.4 Cavity** Where an air space is specified in cavity and veneer wall systems, it shall be maintained reasonably clear of mortar fins and droppings so as to prevent mortar from providing a path for conducting water across the cavity and to prevent mortar from blocking weep holes at the base of the wall. **Notes:** (1) This is to ensure proper drainage and to prevent the formation of mortar bridges which would allow passage of moisture across the cavity onto the structural backing wall. (2) The cavity may be kept clean of mortar droppings by bevelling the mortar beds to incline away from the cavity or by other means. Mortar fins which protrude into the cavity space should be trowelled flat to the inner face of the wythe as the wall is constructed. (3) Mortar droppings in the cavity at the wall base can be reduced by using the techniques described in Note 2. To further minimize the potential for obstructing weepholes with mortar droppings at the wall base: sash cord or other material may be used to form the weepholes provided it is not left in place: coarse gravel may be placed at the base of the cavity; wire screen may be positioned one or two courses above the flashings; or, clean-out openings may be left at the base of the cladding adjacent to weephole locations.

Table 3 CIRIA Study of Incidence of Movement and Cracking Recorded in Survey (24)								
Form of Structure	**Categories of Movement**							
	a	b	c	d	e	f	g	h
Loadbearing brickwork	11	8	2	4	5	2	8	7
Brickwork cladding	11	1	0	1	6	3	1	1
Load-bearing blockwork	6	0	0	3	0	0	0	1
Blockwork cladding	5	3	1	5	0	2	2	2
Totals	33	12	3	14	11	7	11	11

Key: Categories of movement or cracking used in the survey

None

a. No noticeable movement.

Slight

b. Sliding along dpc: oversailing of masonry just above dpc with minor disturbance of mortar.

c. Failure of sealant: either squeezing out or tearing of sealant.

d. Cracking of mortar: stepped cracking through mortar which can be repointed.

Moderate

e. Cracking at short return: vertical splitting of masonry.

f. Spalling corners: fracture and loss of masonry or concrete floors often caused by incomplete dpc at corner.

g. Cracked parapets: disturbance of parapet wall or coping, especially at corners.

h. Other cracking: any cracking which may entail more repair than repointing.

Table 4 Influence of Structural Form and Details on Cracking (22)

Vulnerable to cracking	Less vulnerable to cracking
Short return in long straight wall	
Spandrel walls	Long straight walls
Link bridges	Stepped or corrugated facades
Long parapets	Long returns (greater than 900 mm)
Stronger mortars	Simple unbroken shapes
Discontinuous movement joint	Weaker mortars
Discontinuous dpc	Movement joints in walls
Brick slips	Restrained walls
Incompressible joint fillers	Walls under uniform vertical load
Abrupt curtailment of bed reinforcement	Bed reinforcement in walls
Changes of vertical load	
Slender panels between large walls	
Changes in shape, thickness and height of wall	
Eccentrically confined walls	
Bonding to dissimilar material (e.g. concrete)	

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SEISMIC RESPONSE OF URM BUILDINGS

Andrew C. Costley[1] and Daniel P. Abrams[2]

ABSTRACT

A laboratory study investigated relations between cyclic behavior of unreinforced brick masonry piers and walls with dynamic response of flexible-diaphragm building systems. Experimental results are presented from dynamic tests of two reduced-scale, two-story masonry structures. The three-eighth scale systems were excited by earthquake motions on the University of Illinois shaking table. Rocking of slender piers was a preferred mode of response because of continued strength and deformation capacity. Flexural cracking in the lower story influenced natural frequencies, diaphragm displacements, and lateral force distributions.

INTRODUCTION

Due to budgeting concerns, the testing of civil engineering structures is usually limited to static testing of large-scale structures or dynamic testing of reduced-scale structures. The larger structures often used in static tests are more convincing because real construction materials and techniques can be used, but static tests do not accurately model many dynamic effects, such as inertial force distributions or strain rate effects. Shaking table tests are more realistic in these aspects, but are often given little credence by code writers because of overall size and the use of simulated materials and unusual

[1] Graduate Research Assistant, Department of Civil Engineering, University of Illinois at Urbana-Champaign, Urbana, IL, USA 61801

[2] Professor of Civil Engineering, University of Illinois at Urbana-Champaign, Urbana, IL, USA 61801

construction techniques. In order to perform meaningful laboratory experiments, these concerns should be addressed.

Previous work on an unreinforced brick firehouse instrumented prior to the 1989 Loma Prieta earthquake by Abrams and Tena-Colunga (Tena-Colunga, 1992) brought about the current research program. One project goal was to dynamically test buildings with similar structural features of the firehouse, such as perforated shear walls and flexible diaphragms. The full-sized firehouse had already been "tested" during the Loma Prieta earthquake, so a shaking table study on similar structures would add to the data previously collected. Also, since the seismic input intensity could be controlled during the laboratory tests, further limit states, beyond what the firehouse experienced could be examined.

To minimize the concerns about the validity of reduced-scale tests, another goal was to construct the largest reduced-scale buildings possible, using realistic materials and commonly-used masonry techniques. Although not fully documented here, careful attention to detail was enforced throughout the design and construction of the two test buildings.

The purpose of this paper is to briefly outline the experimental work conducted and present some of the experimental results produced thus far. The results presented center around the inelastic, dynamic effects of pier rocking.

EXPERIMENTAL PROGRAM

Two reduced scale buildings were constructed in the University of Illinois' Newmark Civil Engineering Laboratory for the experimental phase (Fig. 1). Configuration and construction of the test buildings were similar to that of an actual instrumented building which was investigated in a previous study (Tena-Colunga, 1992). Although the buildings were reduced scale, approximately three-eighths, no prototype building was intended or should be inferred. The two buildings were designed to be the largest possible under the constraints of the earthquake simulator, and the three-eighths scale was simply the ratio of the story height, 1.09m (43"), to a nominal full size story height of 3.05m (10').

The two test buildings were each composed of four, two-story, unreinforced brick masonry walls forming a box-type structure. Both buildings measured 2.26m (89.1") long, 1.67m (65.8") wide, and 2.42m (95.4") high. The two longer, shear walls (parallel to the direction of testing) were perforated, and the two shorter, transverse walls (perpendicular to the direction of testing) were solid. The layout of the openings in the shear walls, windows and doors, was varied in order to produce piers with several different aspect ratios. First story pier aspect ratios and cross-sectional areas are listed in Table 1. One test building, S1, had walls with two door and three window openings, while the other building, S2, had walls with three door and two window openings (Fig. 2).

Wall	Exterior Piers		Interior Piers	
	h/L	Area (mm^2)	h/L	Area (mm^2)
S1 Door	1.8	41,300	1.2	64,400
S1 Window	1.9	22,700	1.3	32,000
S2 Door	3.4	22,700	2.4	32,000
S2 Window	1.0	41,300	0.67	64,400

Table 1. First Story Pier Aspect Ratios and Areas.

The bricks used in the construction were 94mm (3.70") long, 45mm (1.76") wide, and 28mm (1.09") thick, and were saw cut from donated pavers. A weak mortar was used, Type O (cement:lime:sand in 1:2:9), to be consistent with older style construction. Horizontal and vertical mortar joints were approximately 5mm (3/16"). All walls were two wythes thick, 94mm (3.70"), and were laid up in American bond, with one header course after five stretcher courses. Careful attention was paid to the bricklaying to produce as realistic a structure as possible (Fig. 3).

Each test building incorporated steel floor systems which were designed to simulate the flexible timber diaphragms common in older, unreinforced masonry structures. Each diaphragm consisted of eleven steel beams supporting ten mass plates and weighed approximately 22.2kN (5 kips). The sizes of the beams and mass plates were set such that the isolated diaphragm would resonate at a frequency much lower than that determined for the masonry walls. These diaphragms spanned between the two perforated shear walls, making them the bearing walls as well. The beam ends were "pinned" into the shear walls and the end beams were tied to the transverse walls with axial links.

Material strengths and construction procedures are documented elsewhere (Costley, 1995). Both buildings were instrumented with almost 40 channels of accelerometers, displacement transducers, and strain gauges. Using the Newmark Laboratory's earthquake simulator, the two test structures were tested dynamically by subjecting them to simulated earthquake motions of increasing intensity. The ground motions were based on the Nahanni earthquake of December 23, 1985.

MEASURED RESPONSE

The two test buildings, S1 and S2, were subjected to a total of nine earthquake simulations (test runs) and ten free vibration tests. Displacements and accelerations of the shear walls and diaphragms at the two floor levels are used in the first four following sub-sections to discuss base shears and drifts, natural frequencies, deflected shapes, and lateral force distributions of the buildings. As an example, the first-level, door-wall displacement from Test Run 13 is plotted at the top of Fig. 4. Vertical displacements measured across a horizontal flexural crack are used to discuss rocking in the fifth subsection.

Base Shear-Drift Envelopes

Inertial force histories were computed from six acceleration histories (one from each shear wall at each floor level and one from each diaphragm), multiplied by the masses tributary to regions of the structure where the accelerometer was mounted. These six inertial force histories were summed to produce a base shear history for each test run. The computed base shear history (normalized by the weight of the building) for Test Run 13 is plotted at the bottom of Fig. 4. Note that the mass distribution for both buildings was approximately 3:5 for masonry mass : diaphragm mass. The diaphragm component of the base shear was, therefore, usually the dominant component. Shear wall displacements from the first level were converted to story drifts by dividing by the story height. The peak drift for each wall was singled out and the two values were averaged to produce a drift index. The peak base shear, divided by the total weight of the building, is plotted against this drift index for each test run in Fig. 5. Limit states are noted in the figure.

The shear-drift envelopes in Fig. 5 give an overview of the dynamic response of S1 and S2. S1 remained linear through the first three test runs (11, 12, and 13), started to crack during Test Run 14, and continued cracking while still resisting substantial lateral loads in Test Run 15. S2 was linear only during Test Run 21, started to crack during Test Run 22, and continued cracking while increasing in load resistance during Test Run 23. During Test Run 24, the peak base shear declined only slightly from Test Run 23. Note that the intensity of the base motion was increased by 1½ times between each of the later test runs. Under continually increasing demand, both buildings continued to resist large lateral forces and demonstrated a substantial deformation (ductility) capacity after the onset of cracking. This ductility was due largely in part to the rocking behavior of the piers.

Natural Frequencies

Fast Fourier transforms were computed from time histories recorded during the earthquake simulations and the free vibration tests. Transforms were taken of data collected by accelerometers, LVDTs, and strain gauges. By examining the transforms of multiple data channels, the dominant frequencies were obtained for each test run. Plots of natural frequency versus drift index for S2 are shown in Fig. 6. Frequencies determined for the free vibration tests are plotted against the peak drift from the previous test run. Limit states are again shown in the figure.

Note that as drift, and hence structural damage in the form of cracking, increased, the natural frequency of the building decreased. This was due not only to the decreased stiffness caused directly by the flexural cracking, but also was due to the inelastic, rocking behavior that developed. The presence of inelastic behavior tends to reduce the resulting structural stiffness and therefore reduces the natural frequency. Also noticeable in Fig. 6 is that free vibration frequencies were higher than those determined from the earthquake simulations. Since free vibration testing was performed with much smaller amplitudes than the dynamic test runs, frequency determination must be amplitude dependent. This is likely because large-amplitude behaviors, such as rocking and sliding, were not induced in the small-amplitude free vibration tests.

Deflected Shapes

Six floor-level displacements, two for each shear wall and one for each diaphragm, were measured relative to the base of the buildings. Deflected shapes were produced by plotting the six deflections at the time of the second-level diaphragm peak displacement. The deflected shapes for two of the S1 test runs, 13 and 15, are shown in Figs. 7 and 8. Note that during Test Run 13, S1 was uncracked, while during Test Run 15, substantial cracking had already occurred. Also plotted in the figure are the averages of the two shear wall displacements (two filled squares), which serve as a reference for the diaphragm deflections.

There were two major differences in the deflected shapes from the uncracked and cracked test runs. The first was that after cracking, the amplification of wall displacements by the diaphragm was greatly diminished. This can be seen by the ratio of total diaphragm displacement to the average of the shear wall displacements. The second major difference after cracking was that the second-story drift had been reduced. Prior to cracking, the wall displacements were almost linear, but after cracking, the second-level wall displacements were noticeably less than twice those of the first level. This effect was also due largely in part to the rocking behavior the first-level piers. Once cracking had been initiated, most of the additional deflection occurred across the cracks, rather than in the undamaged masonry.

Although each of the deflected shapes shown is just for one instant in time, a study of the deflected shapes from the entire test run confirmed that these shapes were representative of the dynamic behavior of both buildings. An examination of all the test runs indicated that for the uncracked structures, diaphragm amplification of wall displacements ranged from 2.5-4.5, while after cracking it dropped to 1.3-1.8. Similarly, for interstory drifts, before cracking second story drifts were 70-80% of first story drifts, while after cracking they dropped to only 10-20%.

Lateral Force Distributions

Floor-level forces were determined using the same six inertial forces (two for each shear wall and one for each diaphragm) that were summed to produce the base shear. This time, however, the three inertial forces at each floor level were summed to produce two forces, or a force pair. A representative force pair from each of the S1 test runs is plotted in Fig. 9. The forces are again normalized by dividing by the weight of the building.

The most striking feature of the force pairs in Fig. 9 was that in each case, the two forces were nearly equal. The force pairs did not follow the linear pattern that is commonly assumed for the lateral distribution of earthquake forces. This behavior could be expected during the cracked test runs, 14 and 15 in this figure, since the upper portion of the building (including both diaphragms) remained undamaged above the rocking first-story piers. The same behavior could be expected for a system of rigid walls and flexible diaphragms since each diaphragm would receive equal input motions. During the uncracked test runs, the masonry walls must have been stiff enough relative to the diaphragms to mimic this behavior.

As with the deflected shapes, the force pairs shown in Fig. 9 are for a single instant in time during each test run. Since these forces were based on rapidly changing accelerations, they too tended to vary rapidly. However, an examination of all the force pairs from an entire test run indicated that on the average, the second floor-level force was equal to the first floor-level force.

Rocking Displacements

Vertical displacements were measured across horizontal cracks at the tops of two of the S2 door-wall piers. By multiplying the vertical displacement histories by the aspect ratio (height/length) of the respective piers, a measure of the (horizontal) rocking displacement can be made. The measurements were taken such that a negative value indicated the crack opening when the building deflected in the negative direction. A portion of one of the rocking displacement histories is overlaid on the first-level wall displacement in Fig. 10. These results were from the left central door pier during Test Run 24.

For the case presented, the rocking displacement was clearly the predominant component of the total first-level deflection. The rocking displacement values were divided into the total displacement values and these ratios were averaged over the strong motion part of the test run. The average for this test run indicated that 80% of the horizontal displacement was attributable to pier rocking. Similar results, though not presented here, were found for S2 during Test Run 23.

SUMMARY AND CONCLUSIONS

Two reduced-scale, unreinforced brick masonry structures were constructed and tested on the University of Illinois shaking table. Each test structure incorporated two perforated shear walls coupled by a flexible floor diaphragm at two levels. The purpose of the tests was to determine the effects of different pier aspect ratios on the dynamic behavior of flexible diaphragm building systems. After nine earthquake simulations, the five following conclusions were drawn.

a) Continued strength and deformation capacity after substantial cracking existed in both test structures.

b) Natural frequencies of the test structures steadily declined with an increase in damage.

c) Flexural cracking in the first stories of the two test structures resulted in reduced diaphragm amplification of wall motions and reduced second-story drifts.

d) Dynamic force distribution between the first and second levels was nearly equal for both undamaged and damaged test runs.

e) After cracking, up to 80% of first-story (horizontal) displacements were attributable to pier rocking.

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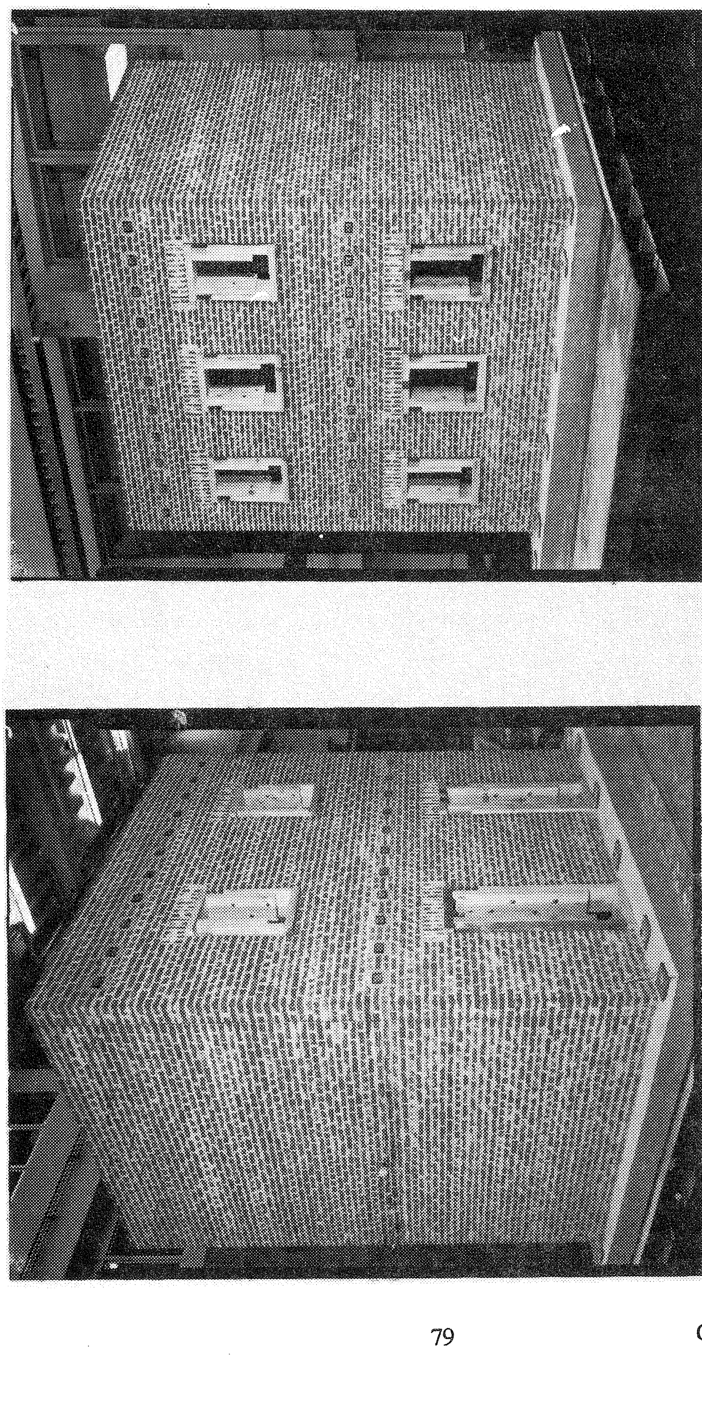

Fig. 1. S1 Door Wall (left) and S1 Window Wall (right) on the Earthquake Simulator.

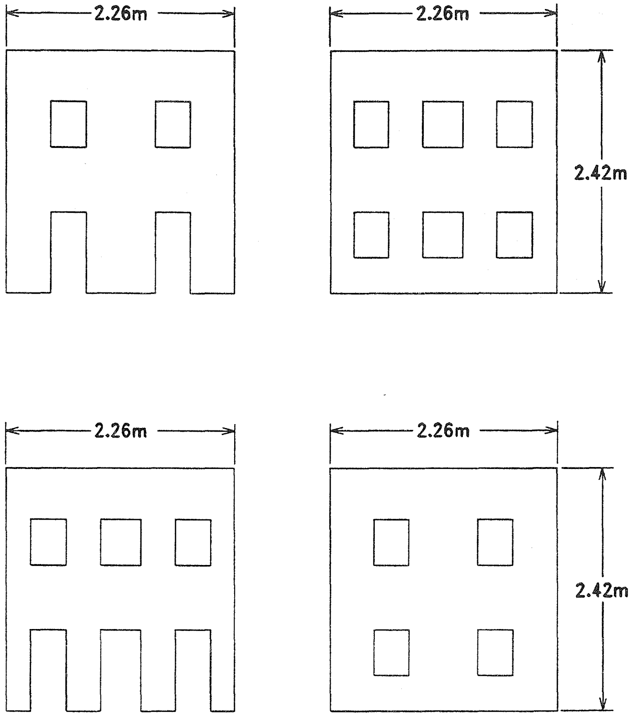

Fig. 2. Configuration of Door and Window Openings for S1 (top) and S2 (bottom).

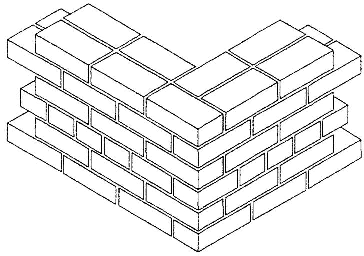

Fig. 3. Corner Detail Used in the Construction of S1 and S2.

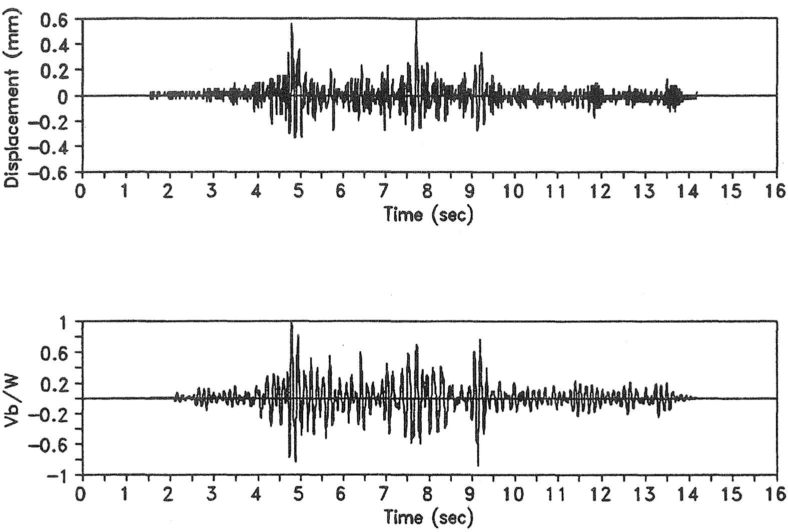

Fig. 4. First-Level Door-Wall Displacement and Base Shear for Test Run 13.

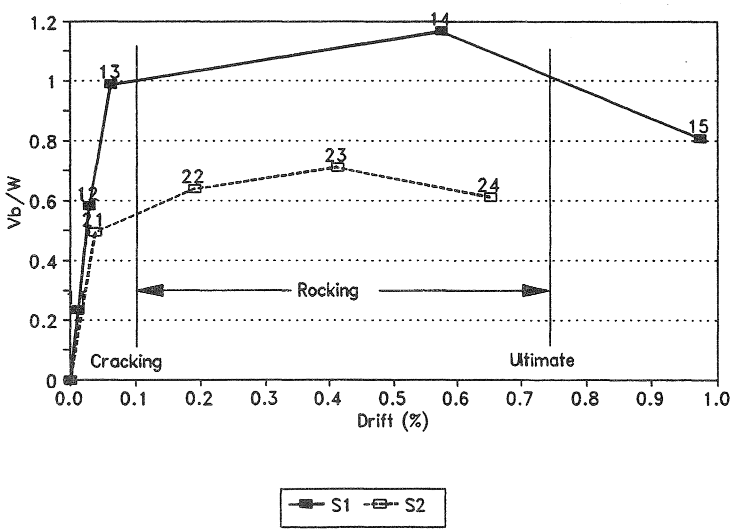

Fig. 5. Normalized Base Shear Versus Drift for S1 and S2.

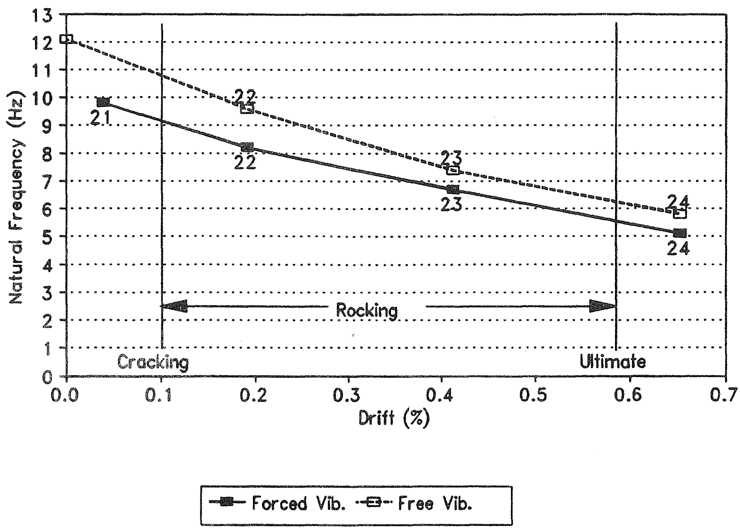

Fig. 6. Natural Frequencies Versus Drift for S2.

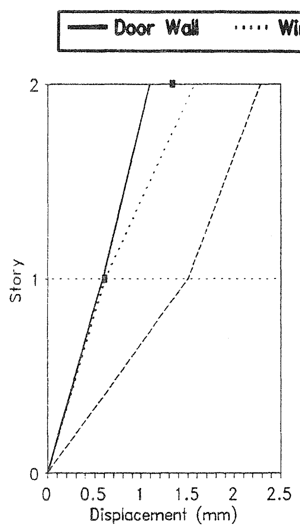

Fig. 7. Deflected Shape for S1 During Test Run 13.

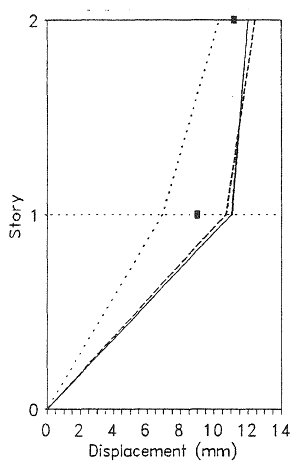

Fig. 8. Deflected Shape for S1 During Test Run 15.