# RESEARCH REPORT

Assessment Repair Strategy for Existing Buildings Constructed with Masonry Veneer Steel Stud Walls

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# Assessment Repair Strategy
# for Existing Buildings Constructed with
# Masonry Veneer Steel Stud Walls

**Prepared for**
**Jacques Rousseau**
**Manager High-Rise Technology**

*Canada Mortgage and Housing Corporation*
*Technical Policy and Research Division*
*National Office*
*700 Montreal Road*
*Ottawa K1A 0P7*

*Prepared by*
*T.W.J. Trestain, P. Eng.*

*T.W.J. Trestain Structural Engineering*
*Toronto, Ontario*

*January 1996*

# *Disclaimer*

This study was conducted by T.W.J. Trestain, P. Eng., T.W.J. Trestain Structural Engineering, for Canada Mortgage and Housing Corporation under Part IX of the National Housing Act. The analysis, interpretation and recommendations are those of the consultants and do not necessarily reflect the views of Canada Mortgage and Housing Corporation or those divisions of the corporation that assisted in the study and its publication.

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This publication is one of the many items of information published by CMHC with the assistance of federal funds.

# Executive Summary

Canada Mortgage and Housing Corporation (CMHC) retained T.W.J. Trestain Structural Engineering to undertake a review of assessment and repair techniques for problem masonry veneer steel stud (MV/SS) walls. This work was motivated by the following:

- Many MV/SS walls have been badly designed and constructed.
- Currently, there is a tendency for designers and contractors engaged in renovations and repairs to these walls to recommend work that in many cases may lead to excessive costs.
- Considerable CMHC sponsored MV/SS research is available to assist with the design and construction of new buildings. There is now a need to transpose this knowledge about new MV/SS construction into the technically more difficult area of rehabilitation of problem walls.

This document is intended to define state-of-the-art "good practice", a needed reference point upon which evaluations of deficiencies, renovation strategies and construction methods can be based.

To answer the question "What style of field investigation?", two competing approaches to field investigation are discussed – the medical model versus the technical audit model. Neither approach is sufficient on its own and a blended model is therefore proposed.

Methods of gathering field information are discussed including a proposed systematic approach to the review of problem MV/SS walls. A compendium of investigative procedures is presented.

The successful diagnosis(es) of problem MV/SS walls requires an understanding of the basic engineering and building science issues. Accordingly, typical MV/SS deficiencies and the consequences associated with those deficiencies are presented.

In order to assist the investigator to answer the question "Is it good enough?", a number of analysis tools are proposed along with some suggested decision making criteria. The analysis tools include reduced Limit States Design load factors, conventional analysis, finite element analysis and full-scale field testing. The basic analysis routine for the finite element program was developed as part of another CMHC project and then customized with a number of features specifically for this project. The program (and worked examples) are distributed on 3 – 1.44 megabyte diskettes and are accompanied by a separate (131 pages) User Reference Manual.

Lastly, a compendium of possible maintain or restore strategies is presented for problem MV/SS walls. Included is a section on economic analysis which provides guidance on the optimum course of action that best meets the needs of public safety, building performance and the financial position of the building owners.

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# Chapter 1 – Introduction

## 1.1 Preface

### 1.1.1 Historical Perspective

During the last 20 years, steel stud/brick veneer has been widely used in Canada as an economical building enclosure system. In 1986, Canada Mortgage and Housing Corporation (CMHC) perceived that the construction of masonry veneer steel stud (MV/SS) walls was proceeding in the absence of the necessary structural and building science knowledge. At that time, they initiated and funded a comprehensive long-term plan for MV/SS research and education. A nation-wide survey was commissioned *(Suter Keller Inc.[1] )* in order to determine the state-of-the-art of MV/SS construction in Canada. This survey helped identify problem areas and define research requirements. The research, undertaken at McMaster University, included structural testing of the steel stud back-up *(Drysdale and Breton[2] )*, brick tie tests *(Drysdale and Wilson[3] )*, testing with temperature, air and vapour pressure differentials *(Drysdale and Kluge[4] )*, and concluded with the full-scale testing of SS/BV wall systems subjected to the simultaneous application of wind and rain *(Drysdale and Wilson[5] )*. This work was complemented by a field survey of eight SS/BV projects that had been built some years earlier *(Suter Keller Inc.[6] )* and a parameter study using a 3-dimensional finite element computer program *(Drysdale and Chidiac[7] )* to examine the influence of crack propagation, openings, corners and intersecting shearwalls.

These research projects culminated in the publication by CMHC of "Exterior Wall Construction in High-Rise Buildings, Brick Veneer on Concrete Masonry or Steel Stud Wall Systems" *(Drysdale and Suter[8] )*. This document is a comprehensive review of the structural and building science requirements for these wall systems. It was intended primarily as an aid to assist with the design and construction of new buildings.

Also during the same period in response to industry needs, the Canadian Sheet Steel Building Institute published the "Lightweight Steel Framing Design Manual" *(Trestain[9] )* which provided detailed guidance on the structural design of the steel stud back-up.

### 1.1.2 Current Requirements

There is now a need to transpose our knowledge about new MV/SS construction into the technically more difficult area of rehabilitation of problem walls. Many MV/SS walls have been and are still being badly built. The widespread incidence of distress and the results of CMHC investigations point to the seriousness of deficiencies in the design and construction of MV/SS.

Currently, there is a tendency for designers and contractors engaged in renovations and repairs to recommend work that in many cases may lead to excessive costs. This is natural and in many ways justifiable given that these firms become liable, at least in part, for the future performance of construction that was not adequate in the first place. This document is intended to define state-of-the-art "good practice", a needed reference point upon which evaluations of deficiencies, renovation strategies and construction methods can be based. This should allow consultants and contractors to move closer to defining the most cost-effective procedures that will insure satisfactory performance while keeping their liability exposure to a minimum. Thus the excessive cost of overly conservative approaches that is seen in some cases can be avoided while at the same time there will be assurance that the minimum requirements are met.

## 1.2. Acknowledgements

## 1.3. Purpose and Scope

This manual is intended to provide building investigators with the currently available techniques for dealing with problem MV/SS walls. These techniques include:

a) methods of gathering field Information
b) decision making criteria to assist with the question "Is this wall good enough and if not what is the appropriate action to take?"
c) strategies for either maintaining, restoring or rebuilding problem walls.

Both building science and structural issues are included with the exception of energy demand considerations, sound control, fire rating and earthquake damage.

The technical information in this manual is intended as an aid to the design professional and should not be used to replace the judgement of a qualified Engineer or Architect.

Note that the assessment and repair of problem MV/SS walls requires familiarity with the basic principles of building science and the structural behaviour of masonry and cold formed steel.

## 1.4. What Style of Field Investigation?

There are two competing approaches to the review of problem MV/SS walls. These two approaches are described here as the medical model and the technical audit model. Neither approach is satisfactory on its own and a blended approach is therefore proposed.

### 1.4.1 Medical Model

In the medical model, the investigating Engineer or Architect essentially responds to and is directed by symptoms of distress observed in the field.

Typically, an investigator would be called to a building site in response to problems reported by building owners and/or users. In addition to making his own visual observations, the investigator might interview the owners or occupants to determine the extent of the problem. The original design drawings would be reviewed to determine the original design intent and one or two inspection openings might be made in order to determine the as-built details.

This preliminary review would likely uncover a number of symptoms all of which suggest a number of possible causes, possible related problems (including safety concerns) and recommendations for further examination and testing.

After the second round of examination and testing, the causes of the observed symptoms would be established along with a recommendation to either maintain, restore or rebuild.

This medical model can be structured in a number of ways. One such method, adopted from an Expert Systems approach, is summarized in Figure I - 1. This

basic structure is expanded in Chapter 2 and presented as "A Systematic Approach to the Review of MV/SS Walls" *(See Item 2.1)*.
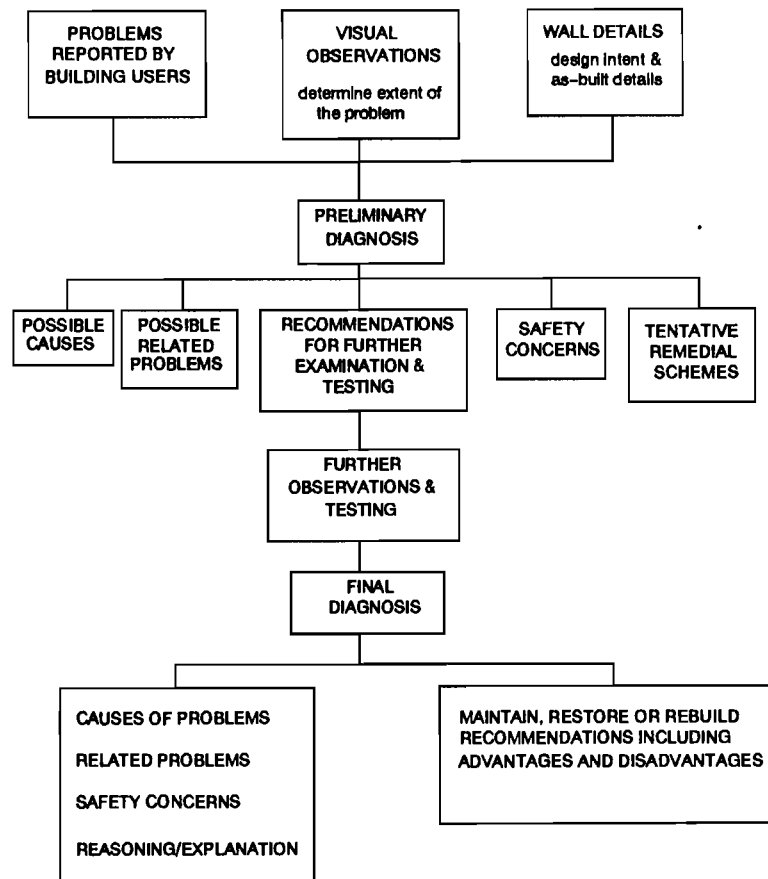


Figure 1 –1 *Flow Chart Illustrating the Medical Model Approach*

### Advantages

- Investigation costs are minimized and unnecessary repairs are avoided. An example of an unnecessary repair might be the installation of vertical control joints (conforming to the requirements of good practice) even though there was no evidence of distress.

### Disadvantages

- There may be some basic problems with the wall but because there are no symptoms, no investigation would occur. This could be particularly serious where deficiencies pose some threat to public safety such as grossly

inadequate steel stud backup or brick ties. The first symptom that might appear in this case is the collapse of the wall in a severe wind storm.

- In the absence of symptoms, deficiencies that have the potential to produce future problems remain unidentified. As a result, there may be no effort to monitor the wall for early signs of distress.

- A purely symptom driven approach may be too complex to document without some expert system software (which does not currently exist) to structure the process.

### 1.4.2 Technical Audit Model

In the technical audit model, the investigating engineer reviews the performance of a MV/SS wall in the context of a performance checklist which includes both building science and structural items. Structural would include the strength of the steel stud back-up, ties shelf angles etc. Building science would include a functioning air barrier, weep holes and vents flashings, etc. See Appendix C for a complete wall performance checklist.

The investigating engineer would go through the checklist and determine where the wall is deficient and what remedial action is appropriate. For building science issues, the absence of symptoms may be sufficient evidence that the wall is performing adequately. Additional evidence would be required to satisfy structural concerns.

See also Torok[10] for more information on the technical audit.

### *Advantages*

- Deficiencies receive attention even if no symptoms are in evidence. This is particularly important for structural issues where a threat to public safety exists.

- Deficiencies that do not require immediate remedial action can be flagged for long-term monitoring.

- Provides a simple, logical structure for field review that is easily documented.

### *Disadvantages*

- Can lead to unnecessarily expensive field investigations and may trigger unnecessary repairs.

- Owners may be reluctant to ask engineers to investigate a particular symptom of distress if standard practice were to require a complete technical audit.*

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* *Currently, few investigators would consider that "standard practice" automatically requires a complete technical audit.*

### 1.4.3  Blended Model

Some blend of the Technical Audit Model and the Medical Model is likely the right approach on any particular project. The investigator should consider a number of factors before deciding what blend is the most suitable.  For example, if the building is covered by a warranty, then the technical audit model is more appropriate. On the other hand, if the owner has provided a limited budget and a limited mandate for investigation, then the symptom driven medical model might be more appropriate except where structural safety is a concern. Issues of structural safety are best dealt with using the technical audit model so that deficiencies receive attention even if no symptoms are in evidence.

| *TABLE 1 - 1*<br>Medical Model or Technical Audit Model? | |
| --- | --- |
| **Are Warranties in Effect ?** | **Appropriate Model** |
| YES | Use the Technical Audit Model. |
| NO | Use the Technical Audit Model for issues of structural safety. |
| | Use the Medical Model for building science issues and structural serviceability issues. |

### 1.5.  Is Every MV/SS Wall Suspect Because It Is MV/SS?

Older steel stud brick veneer walls may be suspect because the necessary structural and building science knowledge was not widely known at the time they were built. In a 1986 nation-wide survey of MV/SS wall design and construction practices (*Keller*[1]),  a number of practices that we now know are deficient were reported as relatively commonplace. For example, exterior insulation was not used on two thirds of the projects reported, 72% of the brick ties did not connect directly to the stud and on 22% of the projects, corrugated brick ties were used.

In addition, based on the experiences of the writer, many MV/SS walls (both old and new) are suspect because this structural system has been subject to an unusually high rate of neglect during design and construction – neglect which is frequently compounded

by a lack of field review by code officials and design professionals.

In the opinion of the writer, it is appropriate to treat MV/SS walls as suspect particularly if the original design drawings and specifications are deficient and/or steel stud shop drawings were not prepared. Under these circumstances, inspection openings should be made to determine the as-built conditions so that the basic structural integrity of the wall can be assessed.

*References*

1      *Brick Veneer/Steel Stud Wall Design and Construction Practices in Canada, by Suter Keller Inc., for Canada Mortgage and Housing Corporation, March 1986.*

2      *Strength and Stiffness Characteristics of Steel Stud Backup Walls Designed to Support Brick Veneer, by R.G. Drysdale and N. Breton, McMaster University, for Canada Mortgage and Housing Corporation, December 1991*

3      *A Report on Behaviour of Brick Veneer/Steel Stud Tie Systems, by R.G. Drysdale and M.J. Wilson, McMaster University, for Canada Mortgage and Housing Corporation March 1989.*

4      *Performance of Brick Veneer Steel Stud Wall Systems Subject to Temperature, Air Pressure and Vapour Pressure Differentials, by R.G. Drysdale and A. Kluge, McMaster University, for Canada Mortgage and Housing Corporation, May 1990.*

5      *Tests of Full Scale Brick Veneer Steel Stud Walls to Determine Strength and Rain Penetration Characteristics, by R.G. Drysdale and M. Wilson, McMaster University, for Canada Mortgage and Housing Corporation, July 1990.*

6      *Field Investigation of Brick Veneer/Steel Stud Wall Systems, by Suter Keller Inc., for Canada Mortgage and Housing Corporation, Ottawa, November 30, 1989.*

7      *Defining Better Wall Systems, by R.G. Drysdale and S. Chidiac, for Canada Mortgage and Housing Corporation, May 1989. (To be Published).*

8      *Exterior Wall Construction in High Rise Buildings, Brick Veneer on Concrete Masonry or Steel Stud Wall Systems, by R.G. Drysdale and G.T. Suter, for Canada Mortgage and Housing Corporation, 1991.*

9      *Lightweight Steel Framing Design Manual, by T.W.J. Trestain, Canadian Sheet Steel Building Institute, July, 1991.*

10      *The Technical Audit Report, by G. Torok, Proceedings, Sixth Conference on Building Science and Technology, Toronto, March 1992.*

# *Chapter 2 – Methods of Gathering Field Information*

This chapter outlines a recommended approach for the systematic review of MV/SS walls and provides a compendium of investigative procedures.

## 2.1 A Systematic Approach to the Review of Problem MV/SS Walls

The Systematic Approach as presented here is based on the flow chart from Chapter 1 *(Fig. 1 - 1)*, a paper by Nicastro[1] where he proposed the use of the scientific method as a useful way to structure water leakage investigations and a paper by Kudder and Lies[2] on diagnosing window and curtain wall leaks.

The method has been generalized to include all types of investigations using the "Blended Model" proposed in Chapter 1.

It is assumed that symptoms of distress are being investigated in order to establish the correct diagnosis(es) for a problem wall. The diagnosis(es) may include building science, structural serviceability and/or structural safety issues. As discussed in Chapter 1, the symptom driven approach may not be enough to resolve all concerns with respect to structural safety and additional investigation in the absence of symptoms, the Technical Audit Model, may be required.

The Systematic Approach can be structured as follows:

- Initial Observations
- Preliminary Diagnosis
- Further Observations and Testing
- Final Diagnosis

with one additional item for structural safety

- Investigate Structural Safety Issues for Which no Symptoms Were Observed

### 2.1.1 Initial Observations

#### a) Locate Areas of Distress by Visual Observation

This is the initial stage of the investigation where the symptoms of distress are located for later correlation with building construction details. (An accurate diagnosis is frequently based on the correlation between observed areas of distress and construction details.)

Both interior and exterior damage is best documented on schematic drawings of

exterior building elevations. See Figure 2 – 1 for an example of typical field notes.
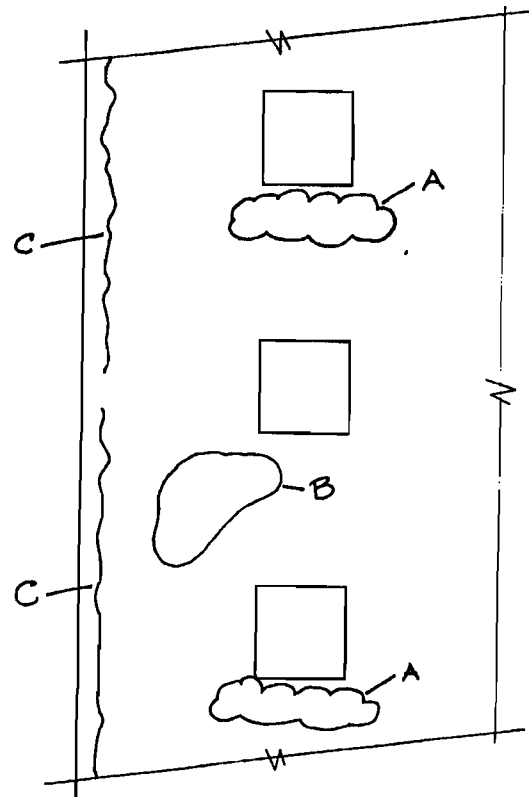


A - *Interior water damage*
B - *Efflorescence*
C - *Corner cracking*

*Figure 2 - 1   Part Elevation of a An Exterior Wall Corner Condition (with field notes)*

Interior damage can be determined by a walk through of the building and interviewing tenants/owners and maintenance staff. To view exterior damage in some detail, scaffolding or a swing stage might be required. Alternatively, binoculars from the ground level might suffice. Many investigators use Infra-red thermography at this stage of the investigation as a "prospecting" tool for locating areas that require further investigation. (See Item 2.4.4 for additional information on thermography.)

### b) Categorize Different Types of Distress

Leaks at window heads might constitute a category as might vertical cracks at corners. Categorizing is a cost effective measure to minimize subsequent investigation where further observation and testing is warranted. Rather than investigating each and every location of distress it is better to examine representative

samples of each type.

### c) Review the Design Intent

Problems can be a direct result of design deficiencies, drawing and specification deficiencies, or due to differences between the design intent and the as-built condition. The primary objective of the design review is to gain familiarity with the design and check key elements and requirements of the specifications and detailing. In addition, deficient design drawings and/or the absence of steel stud shop drawings may indicate the need to investigate a number of structural safety issues even though no related signs of distress have been observed.

If available, the following documents can be helpful:

- Architectural, structural, mechanical drawings
- Project specifications
- Shop drawings
- Construction review reports, change orders, project correspondence

### d) Review the As-built Details

The as-built details can vary significantly from the design drawings and specifications.

For example, if a project suffers budget problems during construction, steel stud might be substituted as an economical alternative to another backup such as concrete masonry. This substitution often occurs after the main structure is up and long after the contract drawings and specifications have been completed. Design details for the steel studs and for air and vapour barriers, flashings, brick ties, vents, etc. are often sorted out at the time of construction and there may or may not be any supporting documentation.

Under most circumstances, inspection openings are the only reliable information source for the as-built details. A number of recommendations for making inspection openings either from inside or outside the building are presented in Item 2.2.2. As-built drawings and photographs taken during construction can also be useful when available.

### e) Review Previous Work

Review previous reports by consultants and/or any previous field work that may have been undertaken to alleviate the problem. Note that if attempts at repair have already been undertaken, it is useful to identify these and other areas of the building that are still in original condition for later correlation with observed distress.

## 2.1.2 Preliminary Diagnosis

The goal of this phase is to develop a comprehensive list of potential causes (diagnoses) so that they may be confirmed by subsequent examination and testing. These causes should be prioritized according to the likelihood of each source.

Establish a correlation between observed areas of distress and construction details. This is the single most important step leading to an accurate diagnosis. Do observed areas of distress occur in a consistent location relative to some feature of the wall? For example, vertical cracks occurring in the brick veneer at building corners might be due to the accumulation of vertical stresses due to discontinuity of the shelf angle or due to the accumulation of horizontal stresses due to the absence of vertical control joints in the brick. The construction details would determine which diagnosis is correct. If insufficient information is available, then the diagnoses would be prioritized and the necessary investigations undertaken in the next phase.

Another useful diagnosis technique is to establish a tree of diagnostic possibilities. For this approach, the diagnostic process is essentially one of elimination – the ruling out of branches that do not apply. See Figure 2 – 2 for an example of a partially completed tree of possibilities relevant to a water leakage problem. This approach is most appropriate for problems, such as water infiltration, where one symptom has a large number of possible causes.
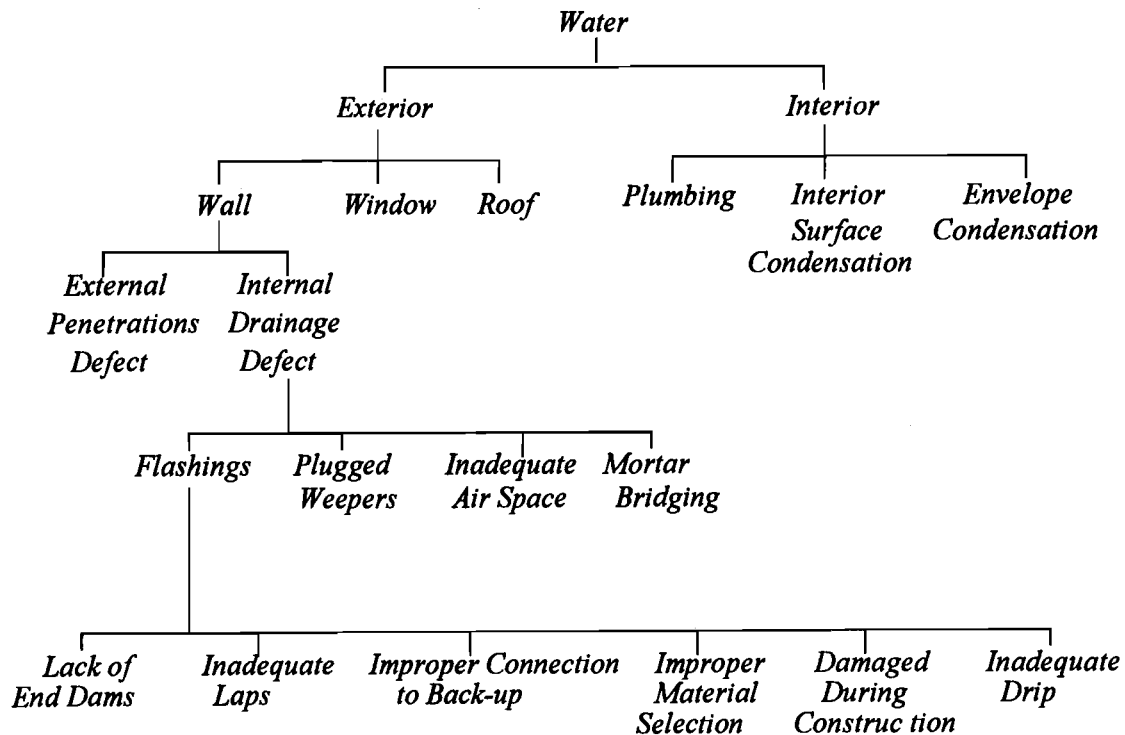
*Figure 2 - 2*

To further assist with the process of diagnosing wall problems, a compendium of symptoms and diagnoses is provided in Chapter 3.

At the conclusion of the preliminary diagnosis phase, there are a number of possible courses of action:

a) There is a high degree of certainty that the preliminary diagnosis is correct and no further examination or testing is required. The preliminary diagnosis is in effect the final diagnosis.

b) Further examination and/or testing is required to confirm the tentative diagnoses. The prioritized list of possible causes will be used to direct the subsequent work.

c) There is a sufficient concern regarding structural safety issues that immediate action is required to protect public safety. The usual safety concern is falling brick and some portion of the ground area below might have to be fenced off from public access.

### 2.1.3 Further Observation and Testing

During this phase of the investigation, additional observation and testing is

undertaken with the goal of eliminating incorrect diagnoses from the prioritized list of possible diagnoses.

Observations to date have been primarily visual from both outside and inside the building. These preliminary observations are necessarily wide-ranging, to establish the scope needed for later, more systematic observation.

More sophisticated (and expensive) investigative techniques are required during this next phase and these are presented under Item 2.2, Compendium of Investigative Procedures. These procedures have been indexed for easy reference in Table 2 – 1.

The scope of the investigations should be limited to keep costs under control but sufficient to lead to an accurate final diagnosis. A number of principles can assist the investigator in this regard:

### a) Statistical Sampling

It is sufficient to examine representative samples of each category of distress. This principle may seem obvious but is sometimes offended when legal action is threatened and it is assumed that redundant information somehow results in a stronger case.

### b) Location of Investigations *(from Trechsel[3])*

Carrying out an investigation at a single wall location is rarely sufficient.

If symptoms are localized, there is an inclination to investigate only at that location. However, the cause of the problem may be more general and symptoms may appear with only a short delay at other locations. More extensive investigation is often warranted.

An all-pervasive symptom might indicate that the same condition occurs throughout the building. Investigating one location would seem to be sufficient except that there is no assurance that the same cause is responsible for the problems at all locations. With a more extensive investigation, this uncertainty can be resolved.

In all investigations, it is also worth considering checking locations where no symptoms have been observed. In spite of the absence of symptoms, it is likely that a number of deficiencies will be uncovered. This information can be useful when studying problem areas – to help isolate the deficiencies that are causing problems from those that are not.

### c) Frequency and Duration of Investigations

Field information can be gathered at a discrete moment in time, periodically or continuously.

Periodic or continuous information gathering is defined as monitoring in the context of this report. Monitoring can be used to determine the rate of deterioration of wall components* or can be used as a means to arrive at a diagnosis for a problem wall.

Monitoring as a diagnostic tool is particularly useful for water leakage problems. Kudder and Lies[2] suggest the following questions when investigating the conditions under which leaks occurs. Monitor to determine the following:

- Are the leaks a one time occurrence under exceptional conditions or are they recurring?
- Do leaks occur when it is raining? Do they occur at the start of the rain or some time later? Do they end with the end of the rain or do they linger?
- Do leaks occur with every rain or only with severe rain or a certain wind direction or velocity?
- Do leaks occur during cold weather?
- Do leaks occur under certain conditions of building operation?

The goal of monitoring is to establish a correlation in time between observed distress and events that may cause the distress. (This is analogous to the previous discussion of establishing a correlation in space between observed areas of distress and construction details.)

For more information on monitoring techniques, see Lawton[4], Keller[54] and Quirouette[5B].

### d) Experimental Remedial Work *(from Kudder and Lies[2])*

In some cases the cost of diagnostic procedures may not be warranted. One possible alternative to testing is to conduct experimental remedial work where if the remedy is successful in eliminating the symptoms then the source of the problem is identified. For this approach to be appropriate, there must be a high probability that the remedy will be successful.

### 2.1.4  Final Diagnosis

To arrive at a final diagnosis, incorrect diagnoses are eliminated from the list of possible diagnoses. This elimination is based on the results of the observation and testing undertaken in the previous phase of the investigation. At this point, the final diagnosis(es) should be established with reasonable certainty and remedial

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\*    *Monitoring to determine the rate of deterioration is one of the possible methods for dealing with problem MV/SS walls. The problem is assumed to be diagnosed, no immediate action is deemed necessary but there are concerns about the long-term durability of the wall or wall component. Monitoring, in this context, can also be used to check the viability of a repair procedure - has the rate of deterioration slowed down significantly?*

work can proceed.

Note, however, that no amount of investigation *guarantees* a correct diagnosis(es). This is an inherent characteristic of any scientific endeavour where experimental evidence is gathered to support or disprove a hypothesis.

### 2.1.5 Investigate Structural Safety Issues for Which no Symptoms Were Observed

As discussed in Chapter 1, it may be necessary to investigate issues of structural safety even when no symptoms are in evidence. This type of investigation typically focuses on the steel stud back-up system and the brick ties particularly for suspect projects that are poorly documented at the design and construction phase.

If inspection openings were made as part of the Initial Observations phase to determine the as-built conditions (Item 2.1.1 (d)), then enough information may be available to assess the structural integrity of the back-up system. If not, inspection openings may still be required for this structural purpose alone.

### Compendium of Investigative Procedures

The format of this section in addition to much of the content has been taken directly from "Review of Non-Destructive Test Methods for Assessing Strength, Serviceability and Deterioration in Buildings" by A.H.P Maurenbrecher and G. Pernica[6]. Additional material has been added including recommendations for inspection openings, more miscellaneous tools, use of surveying instruments, steel and zinc coating measuring techniques, tensile testing and chemistry of steel products, a complete new section on air leakage measurement, a substantially revised and expanded section on water leakage measurement and new section on water vapour transmission measurement.

The investigative procedures presented here are by and large non-destructive - defined broadly as a means of evaluating a wall without altering or damaging it in any significant way. It includes methods which cause either no damage or minor damage that can be easily repaired *(Maurenbrecher and Pernica[6])*. In-situ testing is emphasized. Where laboratory testing is required, suggestions are provided for removing test specimens so that the structural integrity of the wall system is not significantly impaired.

These procedures are summarized for easy reference in Table 2 - 1.

| TABLE 2 - 1 *Index - refer to item numbers indicated* | | | |
|---|---|---|---|
| **Wall Component** *(alphabetical order)* | **Geometric Properties and Composition** | **Strength, Physical Properties and Integrity** | **Moisture, Air Movement and Thermal Performance** |
| Air barrier | *See 2.2.2* | *See 4.3.2.7* | *See 2.4.1, 2.4.2, 2.4.4, 2.4.8* |
| Air space | *See 2.2.2, 2.2.5* | | *See 2.4.3, 2.4.7* |
| Brick veneer | *See 2.2.2, 2.2.4.1, 2.2.4.10, 2.2.4.11, 2.2.4.14, 2.2.4.18, 2.2.4.21, 2.2.5, 2.2.6* | *See 2.3.1, 2.3.2, 2.3.3, 2.3.4, 2.3.6, 2.3.9, 2.3.10, 2.3.11* | *See 2.2.4.1, 2.4.2, 2.4.4, 2.4.5, 2.4.6, 2.4.7* |
| Bridging | *See 2.2.2, 2.2.4.2, 2.2.4.12, 2.2.5, 2.2.7* | | *See 2.4.6* |
| Control joints | | | *See 2.4.7* |
| Drywall – exterior | *See sheathing - exterior* | *See sheathing - exterior* | *See sheathing - exterior* |
| Drywall – interior | *See 2.2.4.17* | | *See 2.4.2, 2.4.5, 2.4.6, 2.4.7, 2.4.8* |
| Expansion joints | | | *See 2.4.7* |
| Flashings | *See 2.2.2, 2.2.5* | | *See 2.4.4, 2.4.7* |
| Insulation – batt | *See 2.2.2, 2.2.4.5* | | *See 2.4.2, 2.4.3, 2.4.4, 2.4.6, 2.4.7, 2.4.8* |
| Insulation – exterior | *See 2.2.2, 2.2.5* | | *See 2.4.2, 2.4.4, 2.4.6, 2.4.8* |
| Lintels | | | *See 2.4.4, 2.4.7* |
| Mortar | | *See 2.3.2, 2.3.3, 2.3.4, 2.3.5, 2.3.9, 2.3.11* | *See 2.4.7* |
| Sheathing – exterior | *See 2.2.2, 2.2.4.4, 2.2.5* | | *See 2.4.2, 2.4.5, 2.4.6, 2.4.7, 2.4.8* |

| Wall Component (alphabetical order) | Geometric Properties and Composition | Strength, Physical Properties and Integrity | Moisture, Air Movement and Thermal Performance |
|---|---|---|---|
| Sheathing – interior | *See drywall - interior* | *See drywall - interior* | *See drywall - interior* |
| Shelf Angle | *See 2.2.3* | *See 2.3.8, 2.3.12* | *See 2.4.4* |
| Studs – built–up at jamb | *See 2.2.2, 2.2.4.2, 2.2.4.7, 2.2.4.12, 2.2.4.16, 2.2.5, 2.2.7* | *See 2.3.12* | *See 2.4.2, 2.4.4, 2.4.6* |
| Studs – typical | *See 2.2.2, 2.2.3, 2.2.4.2, 2.2.4.7, 2.2.4.12, 2.2.4.16, 2.2.4.19, 2.2.5, 2.2.7* | *See 2.3.12* | *See 2.4.2, 2.4.4, 2.4.6* |
| Ties | *See 2.2.1, 2.2.2, 2.2.3, 2.2.4.12, 2.2.5, 2.2.7* | *See 2.3.8, 2.3.12* | *See 2.4.2, 2.4.4, 2.4.6* |
| Track – bottom | *See 2.2.2, 2.2.4.2, 2.2.4.7, 2.2.4.12, 2.2.4.16, 2.2.5, 2.2.7* | *See 2.3.8, 2.3.12* | *See 2.4.2, 2.4.6, 2.4.7* |
| Track – top assembly | *See 2.2.2, 2.2.4.2, 2.2.4.7, 2.2.4.12, 2.2.4.16, 2.2.5, 2.2.7* | *See 2.3.8, 2.3.12* | *See 2.4.2, 2.4.6* |
| Vapour Retarder | *See 2.2.2* | | *See 2.4.1, 2.4.8* |
| Window head and sill member | *See 2.2.2, 2.2.5* | *See 2.3.12* | *See 2.4.2, 2.4.4, 2.4.6* |

## 2.2 Geometric Properties and Composition

### 2.2.1 Infra–Red Thermography

Some larger tie types can be located from the exterior using thermography. Interior to exterior temperature differences as high as 50° C are recommended *(Maurenbrecher and Pernica[6])*. See Item 2.4.4 this Chapter for more detail on infrared thermography.

### 2.2.2 Inspection Openings

***Openings in the interior drywall*** allow the following observations to be made:

- Steel stud back-up member sizes, steel thickness, spacing, connections and any corrosion that may be present
- Bridging and bridging connection details.
- The size and spacing of the fasteners connecting the bottom track to the floor slab (The fasteners connecting the top track to the soffit of the slab above may be hidden depending on the connection detail used.)
- Moisture conditions inside the wall
- The condition of the batt insulation
- The condition of the exterior drywall (batt insulation is usually easily removed)
- The location of brick ties can sometimes be deduced by the presence of sheet metal screws through the exterior flange of the stud (provided the ties were attached with different sheet metal screws than those used for the exterior sheathing and/or insulation).
- Steel samples can be removed for testing.

The interior drywall should be cut to allow easy repair. Vertical cuts in the interior drywall should be made along the centreline of stud flanges so that the remaining drywall and the repair piece both have a surface for bearing and connection. Usually an opening size at least 2 stud spaces wide and floor to ceiling in height works best. Smaller openings are not recommended since they cost about the same as larger openings, create the same disturbance for occupants and generally make it more difficult to examine the inside of the wall. Note that it is usually necessary to repaint the entire wall after an inspection opening has been repaired.

*Openings in the exterior drywall and/or insulation* can be made from inside the building with the interior drywall and batt insulation removed. Removal of the exterior sheathing allows access to the cavity to inspect size, spacing and condition of brick ties, mortar bridges, quality of mortar joints, size of air space, amount of exterior insulation, the condition of the inside face of the brick veneer and quantity of moisture in the cavity. The condition of the flashings and shelf angles are usually difficult to observe. The portion of brick ties embedded in mortar can be examined by chipping away the surrounding mortar to the required depth.

Openings in the exterior drywall made from inside the building are usually difficult to repair and are best kept to a minimum size. A double thickness drywall patch, with outer layer matching the opening size and the inner layer larger than the opening size works well. The patch is glued and caulked in place.

*Openings in the masonry veneer* are made from the exterior side. Horizontal cuts are typically made by sawcutting along a mortar joint and vertical cuts by hammer and chisel. Occasionally, it is possible to chip away at the mortar and remove bricks one at a time allowing the bricks to be re-used to avoid colour match problems. Usually, it is difficult to avoid damaging existing bricks, especially if hard mortar is used. These openings are typically made at the level of the shelf angle to allow inspection of the shelf angle, the shelf angle connection to the structure, the condition of the flashing, and whether or not the weepers are blocked by mortar droppings. Wide openings are to be avoided since they can affect the structural integrity of the wall.

### 2.2.3 **Metal Detectors** *(from Maurenbrecher and Pernica[6])*

Most metal detectors are based on electro–magnetic induction for magnetic metals such as mild steel, or on eddy current principles which can also be used to detect non–magnetic, conducting metals. The best and most stable detectors use pulses of current in a coil to create temporary magnetic fields in the immediate vicinity of the coil (pulse–induction eddy–current method). When the current is suddenly switched off, eddy currents are set up in any conductive metallic objects within the field. These currents in turn try to preserve the induced magnetic field. This in turn is detected by the instrument. The method is not affected by moisture.

Metal detectors also use designs based on beat frequency oscillators (the simplest and least expensive type of detector), induction balance (a method used by reinforcement detectors) and VLF phase angle (VLF: very low frequency). These do not have the stability of the pulse induction systems. The latter is therefore used where accurate detection is required.

Wall ties can usually be detected to within 50 mm. Coils for ties should be designed to look near the surface. Most will detect ferromagnetic and non–ferrous metals but some will not detect austenitic stainless steel very well without special search coils and/or special electronics.

See Maurenbrecher and Pernica[6] for more detail

### 2.2.4 **Miscellaneous Tools**

2.2.4.1 ***Binoculars*** for preliminary review of building exterior or for viewing areas inaccessible by swing stage, ladder or scaffolding.

2.2.4.2 ***Callipers (inside and outside)*** for measuring stud flanges lips and depths.
Callipers with digital readout are best.

2.2.4.3 ***Camera and flash*** for still photographs. Useful features include a date stamp capability on the camera and bounce flash. Each photograph should include a ruler for scale and a written description of what is being photographed.

2.2.4.4 ***Coat hanger*** for measuring wall thickness by punching through interior drywall. Exterior drywall layer also be located.

2.2.4.5 ***Crochet hook*** for pulling out a small sample of batt insulation through a small hole in the interior drywall.

2.2.4.6 ***Compass*** for building orientation.

2.2.4.7 ***Electric Drill with 38 mm ± diameter circular cutting tool*** for removing circular steel specimens from studs. Circular specimens of this diameter are useful for checking galvanizing and thickness of steel exclusive of coating. For studs 92 mm and greater in depth a specimen of this size will not significantly impair the structural performance of the stud if taken from the mid-depth of the stud over the middle half of the length of the stud.

2.2.4.8 ***Flashlight*** including large lantern to a small light with fibre optic extension.

2.2.4.9 ***Heat gun (paint stripping variety)*** can be used to dry wet surface areas so that continuing leakage can be observed more readily.

2.2.4.10 ***Level (carpenter's)*** for checking horizontal surfaces (e.g. to check water flow direction on an exterior balcony) or for checking plumbness (e.g. to check brick veneer for bowing which may indicate distress).

2.2.4.11 ***Magnifying glass*** both for viewing the wall and for picking out small detail on photos.

2.2.4.12 ***Micrometer*** for measuring thickness of steel framing products.

2.2.4.13 ***Mirrors*** including shaving mirror to dentist size.

2.2.4.14 ***Plumb Bob*** for checking plumbness over large distances – say one to several floors.

2.2.4.15 ***Pocket knife***

2.2.4.16 ***Radius gauges*** for measuring the inside bend radius of studs. (The inside bend radius is an important parameter where web crippling governs.)

2.2.4.17 ***Stick-on plastic sheets*** can be used to protect interior carpets from dust when drywall is cut for inspection purposes.

2.2.4.18 ***String*** for measuring out-of-straightness.

2.2.4.19 ***Stud Sensors*** will locate steel (and wood) studs. To locate the edges of each stud slide the sensor horizontally over the interior drywall. Steel studs are more difficult to locate when foil backed drywall is in place. These types of sensors are available form most hardware stores for minimal cost ($30.00).

2.2.4.20 ***Tape Recorder*** for recording observations.

2.2.4.21 ***Telescope with tripod and spotting scope*** for use where binoculars do not show enough detail.

2.2.4.22 ***Vacuum (wet and dry type)*** can be used to vacuum dust when drywall is cut for inspection purposes. Ordinary household vacuums are not suitable since the collection bags clog quickly with gypsum dust and the dust can be harmful to the motor bearings.

## 2.2.5  Optical Probe (Borescopes, Endoscopes and Fibrescopes)

The probe consists of a rigid or flexible tube containing optical lenses, mirrors and/or glass fibres enabling an observer to look into inaccessible cavities through small openings. A light source is provided usually through optical fibres. The end of some probes is articulated or lenses can be attached including side viewing ones. A camera or video camera can be attached to the probe eye piece. For viewing the wall cavity from the outside, a tube diameter from 8 – 10 mm will allow access through holes drilled in the mortar joints.

Probes often give only limited information. For example, while tie corrosion in the wall cavity can be observed (assuming mortar droppings do not interfere), corrosion of the portion embedded in masonry cannot.

See Maurenbrecher and Pernica[6] for more detail.

## 2.2.6  Surveying Instruments

***Transits*** set up at ground level can be used to measure the plumbness of a wall. A tape or rod projecting horizontally from the wall can be moved via a swing stage or a  light unmanned carriage can be fabricated and run by ropes from the roof level.

***Measuring Tapes*** includes simple tapes and hand held ultrasonic instruments with digital display.

## 2.2.7  Thickness of Steel Excluding Coating and Coating Thickness Measuring Devices

***Coating Thickness Gauges*** are magnetic gauges suitable for measuring the thickness of zinc on steel. Accuracy is in the order of ±15%. This procedure can be used for purposes of acceptance of galvanized product but not rejection. See ASTM A446[7] and ASTM E376[8] .

***Acid baths*** can be used to strip zinc coating from steel parts. The thickness of the steel exclusive of coating can be measured (by micrometer) directly after soaking in an acid bath. In addition, before and after measurements can give an estimate

of zinc coating thickness. Submerge the part to be measured in a 5% – 10% solution of muriatic acid. Remove the part when the visible bubbling stops. Do not leave the part in the acid bath any longer than necessary to remove the zinc since the acid will attack the steel but at a much reduced rate.

***Single and Triple Spot Tests*** are the formal laboratory procedures (similar to acid baths) for determining the thickness of zinc on steel. Refer to ASTM A525[9] and ASTM A90[10]. The single spot test is the most appropriate for field studies. For test specimens less than 50 mm wide, an area of approximately 3000 mm² is required. Removing a test specimen this size might impair the structural performance of the stud.

## 2.3 Strength, Physical Properties and Integrity

### 2.3.1 Acoustic Impact Testing

By tapping the masonry veneer with a wooden or rubber mallet, internal fractures (delaminations of the brick) can be detected because the damaged brick has a different sound than undamaged brick. The method is very approximate and requires an experienced operator.

### 2.3.2 Bond Wrench Test for Masonry *(From Maurenbrecher and Pernica[6])*

An accurate and simple method of determining the flexural tensile strength of the bond between the masonry unit and mortar (on a bed joint). A long lever is attached to a masonry unit free on all sides except the bottom face which is still attached to the mortar joint in a masonry wall or wallette. An increasing load at the free end of the lever applies a bending moment to the assembly until failure occurs at the joint.

Laboratory tests are conducted in accordance with ASTM C 1072[11] but can be adopted for field use as described in UBC 24-30[12]. The test provides flexural bond strength perpendicular to the bed joint for use in follow-up analytical studies that take advantage of the bending strength of the brick veneer. Requires a temporary opening in the wall that requires subsequent repair. See Maurenbrecher and Pernica[6] for more information.

### 2.3.3 Coring

A power drill is used to cut a core sample from the masonry veneer. The cores can be used to determine salt content and composition. Note that cores cannot be used to determine moisture content of the masonry – a wet coring procedure adds water and the heat generated by a dry procedure drives water off.

### 2.3.4  Crack Gauges

***One-time observations of crack width*** can be made with a variety of measuring devices. Automotive gauges used for measuring spark plug gaps can be used to measure crack widths as small as 0.07 mm.

***Monitoring crack width development*** can be done continuously or intermittently. Cracks may have daily or seasonal changes as well as long-term growth.  Inexpensive plastic tell-tale gauges can be mounted across and left in place for periodic reading. Alternatively, Demec, Whittemore or vernier callipers can be used to measure the change in crack size between readings. Cracks can be continuously monitored by making use of a gauge that records the crack movements as scratches on a replaceable brass button. The scratches are read with a calibrated microscope. Alternatively, electronic gauges such as LVDT's can be used for continuous recording. For more information see Maurenbrecher and Pernica[6].

### 2.3.5  Mortar Sampling

Mortar can be removed with hand tools to test for chloride content in the laboratory.

### 2.3.6  Impact Echo and the Ultra-Sonic Pulse Velocity Methods *(from Maurenbrecher and Pernica[6])*

For the impact echo method, a stress pulse is applied to the surface of the veneer by mechanical impact. The stress wave propagates within the member and is reflected at material discontinuities such as cracks, voids or member boundaries. The reflected waves are measured at the impact point or at a number of different points. For the ultra-sonic pulse velocity method, an electromechanical transducer transmits a high frequency pulse to the member under investigation. Changes in the velocity and amplitude of the transmitted wave can be measured. Both methods have the potential to detect delamination of bricks due to frost damage but should be considered experimental. See Maurenbrecher and Pernica[6] for more information.

### 2.3.7  Load Testing

Load testing of complete wall assemblies is used to predict the real behaviour of the wall assembly and as such is an alternative to analysis. Load testing is discussed in Chapter 4.

### 2.3.8  Pull-Out Testing

The strength of anchors in masonry or concrete can be determined by running a

pullout test in accordance with the requirements ASTM E 488[13].

### 2.3.9  Radar *(from Maurenbrecher and Pernica[6])*

Electromagnetic waves travel through the veneer and are reflected or refracted where the material changes in composition, density or water content. Radar can used to locate ties (including plastic ties), areas of high chloride concentration, voids and delaminations. The equipment is operated similar to a metal detector by one person but considerable experience is required and the equipment is expensive. See Maurenbrecher and Pernica[6] for more information.

### 2.3.10 Strain Relief Testing

Electrical resistance strain gauges are attached to the masonry and readings taken. The portion of brick with strain gauges attached is cut loose from the wall and the gauges are read again – the change in gauge reading is a measure of the strain in the masonry. Multiple measurements allow a stress map to be created for the exterior elevation. This technique can be used to determine if addition of expansion joints will relieve the stress build-up *(Thomassen and Searles[14])*.

### 2.3.11 Surface Hardness Testing *(from Maurenbrecher and Pernica[6])*

Surface hardness testing is measured by the rebound a of a hammer on the surface of the material being tested. The rebound distance is an indication of the compressive strength of the material. A pendulum hammer has been adopted to assess the quality of the mortar in the outer part of the mortar joint. Also, hammer tests can be used to determine the quality of brick to a depth of 50 mm. Refer to Maurenbrecher and Pernica[6] for more information.

### 2.3.12 Tensile Testing and Chemistry of Steel Products

*Tensile tests* for steel components are conducted in accordance with ASTM A 370[15]. It is preferable to send a length of as-rolled product to the laboratory rather than attempting to remove a tensile specimen in the field. Where this is not possible, the following guidelines are suggested:
- Remove specimens from the flat portion of a member. Mechanical properties are distorted near bend radii.
- The longitudinal axis of the test specimen and the member should coincide.
- Cutting procedures should create a minimum amount of heating and cold working of the specimen.
- The minimum standard specimen size is 20 x 200 mm and the minimum subsize specimen is 10 x 100. Subsize specimens can be removed from studs 92 mm and greater in depth without significantly impairing the structural performance

of the stud if taken from the middle of the web of the stud over the middle half of the length of the stud.

*Chemistry tests* (in addition to tensile tests) are required when it is necessary to know if steel conforms to a particular structural standard (such as ASTM A 446). Testing in accordance with ASTM E 663[16] is one of the available laboratory procedures.

## 2.4 Moisture, Air Movement and Thermal Performance

### 2.4.1 Air Leakage Measurement

#### General

Air leakage through the building envelope affects the energy demand to heat and air condition the building, the indoor air quality, and condensation within the building envelope.

Concealed condensation within the wall assembly is critically important to the performance and durability of MV/SS walls (energy demand and indoor air quality are beyond the scope of this report). Moisture accumulation can occur during the winter months due to excessive air exfiltration and/or in the summer months due to excessive air infiltration where the indoor space is air conditioned.

#### 2.4.1.1 ASTM E 1186, Practice for Air Leakage Site Detection in Building Envelopes[17]

#### 2.4.1.2 Fan Pressurization/Depressurization

A method for measuring air leakage into or out of a building space under a controlled pressure difference. Test methods for measuring air leakage rates of the whole building, individual floors and exterior walls of individual apartment units are described in Shaw and McGee[18].

An air flow generator, or fan, creating a positive or negative pressure is installed in a window or door to create a positive or negative pressure in the building space. The volume of air flow is measured at the location of the fan. Results are dependent on the magnitude of the applied pressure. Alternatively the HVAC system can be used to pressurize the building using 100% outside air in the supply air system (provided sufficient pressure difference can be created). See also ASTM E 779[19] for test procedures using the pressurization method.

Note that pressurization techniques do not isolate a specific source of leakage since multiple components are being tested and are more

appropriate for measuring the impact of overall leakage rates on energy demand for the building.

### 2.4.1.3  Infra-red Thermography

Infra-red thermography can be used to locate air leakage sites in MV/SS walls. Note, however, that the presence of the air space in the wall assembly will to some extent mask air leaks through the inner wythe.

If the building is being depressurized by the HVAC system then air leakage problems will not show up. On the other hand, deliberately pressurizing the building with the HVAC system can reveal potential air leakage problems.

See Item 2.4.4 this chapter for more detail.

### 2.4.1.4  Laboratory Procedures

Laboratory tests for air leakage through assemblies are generally conducted in accordance with ASTM E 283[20] and for structural integrity of air barriers in accordance with ASTM E 330[21]).

### 2.4.1.5  Smoke Pencils and Smoke Machines

Smoke pencils are hand-held devices that release a small amount of white chemical smoke-like material when triggered or squeezed. The user can observe the direction and approximate velocity of local air currents. Smoke pencils are available in a variety of types including disposable type, reusable tube type (with refill chemicals that are somewhat messy but less expensive) and gun type with refill chemicals. They are very useful for locating air leakage paths in the building envelope and are a basic tool for air leakage investigations. They do not work well in low temperatures and may have a tendency to clog in very high humidity environments.

Smoke generating machines can be used to fill a room with smoke that has been positively pressurized allowing leakage paths to be observed from the outside.

### 2.4.1.6  Tracer Gas *(from Trechsel[3])*

***Tracer gas dilution method*** is useful for determining air leakage into or air movement within building spaces, including wall cavities. A tracer gas is injected into the space and the decay of the tracer gas is measured over time. ASTM E 741[22] describes this method. Other tracer gas techniques are based on constant injection rates or on constant concentration rates. The use of several tracer gases simultaneously allows the determination of air flow among several building spaces or cavities. At the

present time standardized test methods do not exist for any other than the dilution method.

Tracer gas tests can be run under actual air pressure conditions as they occur in the building but require a competent operator and appropriate electronic equipment. If actual pressure conditions are to be used multiple measurements are desirable spanning different seasons and weather conditions.

See also Harrje et al.[23] for additional information.

### 2.4.1.7  Ultrasonic Leak Detectors

Senses the presence of a crack that might represent an air or water leakage path. An ultra sonic sound signal is sent through the crack area and the return signal is read and interpreted.

Best performance if no pressure differential is present across the crack (in which case a smoke pencil would suffice). Does not detect crack size nor is it a useful technique for cracks with "devious routes".

### 2.4.1.8  Weather Induced Pressure Differences

Air infiltration and exfiltration rates through the exterior wall of an individual apartment unit can be estimated by measuring the pressure difference across an exterior wall under various weather conditions. These pressure values are combined with the measured air tightness value of the exterior wall expressed in terms of the coefficients C and n determined by the fan pressurization method (see Item 3.1.3 this chapter). The air infiltration/exfiltration rate can be determined from the standard flow equation $q = C (\Delta P)^n$. For more information see Shaw and Magee[18].

Note that pressurization techniques do not isolate a specific source of leakage since multiple components are being tested and are more appropriate for measuring the impact of overall leakage rates on energy demand for the building.

## 2.4.2  Condensation Sensors

Condensation sensors are used to measure time of wetness, or detect dewpoint temperatures. One type acts as an electrochemical cell, producing a small voltage when it becomes wet. Another more sensitive type, detects condensation on a mirror. *(Maurenbrecher and Pernica[6])*

For more information see ASTM G84[24] and Serada et al.[25].

## 2.4.3 Humidity Measuring Devices

Humidity can vary significantly inside an air space at any moment in time. It is therefore best to take a number of measurements at various locations.

***Crystal sensors*** are the simplest sensor. Changes in humidity are read as changes in colour of the crystal. See Maurenbrecher and Pernica[6] for more information.

***Household Hygrometer*** is based on a dimensional change of an organic material such as hair with an accuracy of ±3% and a slow response time.

***Electronic Hygrometer*** is a hand–held device with an accuracy of 2 – 5%. Many units are equipped to measure humidity and temperature simultaneously. Some have small (12 – 19 mm) diameter probes for collecting data through openings in wall cavities. Some are equipped with devices to allow the recording of data over an extended period of time. See Maurenbrecher and Pernica[6] and Trechsel[3] for more information.

***Sling Psychrometer*** has two identical thermometers mounted together with a pivoting handle at one end that allows both thermometers to be the whirled at about 60 – 120 rpm. A wet bulb temperature is taken from the thermometer with a clean wetted wick over the bulb. A dry bulb temperature is taken from an identical thermometer with a bare bulb. The relative humidity can be found from these two measurements using a standard psychometric chart. The accuracy at room temperature is ±3% and is dependent on whirling speed. Only distilled water should be used to wet the wick. For further information, see Hutcheon and Handegord[26] and Maurenbrecher and Pernica[6].

## 2.4.4 Infra–Red Thermography *(from Maurenbrecher and Pernica[6] and OBEC Tech Note[27])*.

***Background and principles:***

Infrared thermography is a technology that allows infrared or heat radiation to be transformed into a visible image. Thermographic surveys on walls are typically undertaken from the exterior side.

A sensing device converts infra–red radiation radiated from a surface to a voltage signal. The voltage signal can be converted to temperatures and displayed in the form of a monochrome or colour image. Each colour or shade of grey conforms to a pre–set temperature range. In addition, on some machines isotherms can be generated. Temperatures from –30° C upwards can be detected (depending on the camera). The best equipment can determine temperature with an accuracy of ±1° C and a resolution of 0.1° C. Computer analysis software can be used to analyze the results.

### *Applications and Advantages:*

- Gives information over large areas not just point measurements
- Can be used to detect moisture in brick (areas with moisture are cooler)
- Can identify areas where insulation is missing or displaced (resulting in convective "short circuits")
- Can identify areas of wet insulation
- Detects air leakage, heat loss and thermal bridging
- May be possible to identify incipient spalling in the brick since the surface of the brick is separated by small cracks which act to insulate the brick surface from the warmer mass of the brick
- May be possible to identify tie locations
- Could be used to locate studs and interior drywall surface temperature differentials in order to assess the dust marking potential of the wall system.
- Valuable as a "prospecting" tool for locating areas that require further investigation

### *Limitations*

- Environmental conditions must be suitable for accurate test results including no sunshine on the facade for a minimum of 2 or 3 hours after sunset to allow solar radiation gained by the brick during the day to be lost. On cloudy nights, when radiation is reflected back to earth up to 12 hours may be required.
- A temperature difference of at least 10° C is required across the wall with an optimum of 20° C . An outside temperature near 0° C is therefore best.
- The wall surface should not be visibly wet.
- Temperature variations can be masked by excessive wind and/or excessively low outside temperature.
- Emissivity (the ability of a material to emit radiation) varies with wavelength, material type and surface finish. Differences in emissivity of adjacent materials can cause a "hot" material to appear the same as a "cold" material or vice versa.
- Reflection of light on a surface being scanned can distort images. Measurements therefore have to be done at night. Care is required to avoid the reflection from artificial light sources.
- The operation of the building HVAC systems can dramatically affect the performance characteristics of the building envelope at the time of the scan. For example, night-time temperature setbacks could reduce the indoor/outdoor temperature differential and mask thermal bridging problems. If the building is being depressurized by the HVAC system then air leakage problems will not show up. On the other hand, deliberately pressurizing the building with the HVAC system can reveal potential air leakage problems.
- On MV/SS walls, thermal behaviour of the steel stud back-up will be substantially masked by the insulating effect of the air space between the wythes. For example, some building science practitioners report success locating air leaks through the interior air barrier and others do not. Except in rare circumstances, it is unlikely that studs can be located by exterior thermography.
- There are no widely recognized standards for training thermographers

- Experience indicates that the usefulness of thermography on any particular wall investigation is difficult to predict and that a trial run is best.
- There is a potential for abuse. The wrong equipment, poorly trained thermographer, adverse weather conditions, poor knowledge of the building and ignorance of building science could lead to the collection of misleading information

See Maurenbrecher and Pernica[6], ANSI/ASHRAE 101[28], Burn and Schuyler[29], Chown and Burn[30], CAN/CGSB-149.2-1986[31], and ASTM C 1060[32] for more detailed information.

### 2.4.5 Moisture Meters and Measuring Techniques

*Capacitance type moisture meters* produce a high frequency of the order of 12 MHz to measure the dielectric resistance of the material with which it is in contact. Water can be distinguished from other building materials because it has a much higher dielectric constant. These meters are most useful for giving relative values of moisture but can be used for absolute values if calibrated with the same material. In contrast to the resistance moisture meter, the capacitance moisture meter is not affected by the presence of dissolved salts. Maximum penetration is 30 – 50 mm. For more detail, see Maurenbrecher and Pernica[6].

*Resistance type moisture meters* measure electrical resistance between two electrodes usually in the form of pins. Resistance decreases with increasing moisture levels. This type of meter is most commonly used to measure the moisture content of wood where the pins can be driven into the wood to measure moisture content beneath the surface. The moisture content of other soft materials such as gypsum drywall can also be measured in this manner. For masonry, three approaches are possible: a) readings can be taken under a plastic sheet laid over the surface where moisture levels will approach those further within the member, b) electrodes can be inserted in pre-drilled holes c) a small probe (commonly called a matchstick probe) which in itself carries the electrodes is inserted into pre-drilled holes. For the latter method, the meter measures the moisture content of the probe itself which can be correlated with the moisture content of the masonry. Calibration is required for all methods. For more detail, see Maurenbrecher and Pernica[6].

*Weighing and oven drying* is the most reliable measurement method for moisture content. The disadvantages with this method include the need to remove samples from the wall and the delay inherent in off-site testing.

### 2.4.6 Temperature Measuring Devices

*Digital Thermometer* is best for measuring air temperature because of quick response time compared with conventional thermometers. Note that all thermometers will measure air temperature correctly only if they are suitably shielded from surfaces that are at other than air temperature. For more information, see

Hutcheon and Handegord[26].

***Thermocouples*** work on the principle that two types of metal in mutual contact generate a voltage in proportion to the temperature. These measuring devices are normally associated with laboratory work but can be useful in the field where long-term monitoring is required with automated sampling and recording of temperature measurements. For more information on calibration and set-up problems refer to manufacturers' literature.

***Contact thermometers*** require direct contact between the instrument and the surface whose temperature is required. These temperature measuring devices are useful for quickly obtaining surface temperatures at spot locations.

***Infrared non-contact thermometer "gun"*** with a cone shaped signal allows a larger area to be measured the further from the wall the measurement is taken.

***Infra-Red Thermography*** can be used to obtain thermal profiles over large wall areas. Refer to Item 2.4.4 for more information.

## 2.4.7 Water Leakage Measurement

### 2.4.7.1 General

The basic test procedures for water leakage testing have been developed by ASTM and AAMA. The application of these procedures to in-situ testing of MV/SS wall systems typically requires some modification to the testing practices and/or the pass/fail criteria. Much of the content in this section has been taken from Zwayer and Johnson[33].

Refer to Section 2.4.7.8 for an overview of suggested methodologies for investigating water leakage problems.

### 2.4.7.2 Water Permeability Measuring Devices/Techniques

Permeability may be assessed by measuring the rate of absorption of water per unit area measured either under constant pressure or as the time of absorption for a given quantity of water *(Maurenbrecher and Pernica[6])*.

***Rilem tube*** is a simple device based on the latter approach. A graduated cylinder open both ends with a 90° return at the bottom is attached to the wall with putty. Once filled, the speed with which the water level falls indicates the relative permeability of the wall. This is a highly localized test but can be useful for assessing the relative effectiveness of water repellent treatments, or for checking the permeability of individual masonry units or mortar joints. For further information see Maurenbrecher and Pernica[6].

*ASTM E 514*[34] is a laboratory test procedure with suggested field modifications outlined in ASTM STP 778[35]. A test chamber, usually 600 x 900 or 900 x 1200 is sealed to the outside face of the masonry at typical wall locations. Water spray is applied at a rate of approximately 140 l/m² per hour (equivalent to a rainfall intensity of 140 mm per hour – Zwayer and Johnson[33]. Simultaneously a static air pressure can be applied inside the chamber – using 0.0, 0.25 and 0.50 kN/m² as typical test values . The amount of water lost from the closed loop circulation system is taken as the amount of water absorbed or permeating through the masonry.

It is recommended in Zwayer and Johnson[33] that 3 – 5 wall locations be selected around the building and that they be representative of typical conditions. In addition, it is suggested that the primary value of this test is to establish a rate of water loss through the wall for use in a follow–up drainage test. (See Item 3.7.3 Wall Drainage Test). In order to observe any leakage paths through to the interior, removal of interior and exterior finishes may be necessary.

The disadvantages with this test reported in Zwayer and Johnson[33] include the length of time to obtain steady state conditions (2 – 4 hours) and the surface area exposed to wetting is small.

Brook[36] suggests using the driving rain wind pressure (DRWP) based on climatic data from Environment Canada as a more rational basis for selecting an applied external pressure.

Many investigators report using this test procedure successfully while others are critical of its utility. In Brown[37], it is noted that the field–adapted test procedure has several inherent weaknesses including:
- no way of providing termination of the sample beyond the chamber resulting in adjacent surfaces possibly influencing the test
- no guidance on pre–conditioning – a dry wall will absorb more water during the test than a wet wall.
- water quantity and water pressure are not defined
- by measuring water lost from the closed loop, nothing is learned about where the water goes once it enters the masonry.
- rating system of "expected", "significant" and "excessive", based on various cut–offs of water quantity leaving the closed loop, is arbitrary and unrelated to susceptibility of the wall to water damage.

### 2.4.7.3 Wall Drainage Test

This procedure is described in Krogstad[38]. Water is introduced directly into the interior side of the exterior brick via a tubing system to trickle down the inside face of the masonry and accumulate on the flashings or other obstructions. Interior observations are made during the test to

check for leaks and exterior observations to check the performance of the weepers. The test is particularly useful for evaluating the effectiveness of the air space, flashings and weepers.

***Advantages*** include the ability to check large areas of the wall simply and quickly.

***Disadvantages*** are related to the precision required to run the test. The test results can vary significantly depending on how far into the wall cavity the tubes are inserted. In addition, any flashing system can fail if the rate of water introduction is high enough. It is recommended in Zwayer and Johnson[33] the rate of water introduction be based on the results of the ASTM E 514 water permeability test. See Item 2.4.7.2.

### 2.4.7.4  Water Dyes

Can be used to trace leaks. Dyed water is applied near the suspected leak and any infiltration of coloured water is noted. Fluorescent type dyes are best. Some investigators carry a plastic squeeze bottle of dyed water for small scale local leakage tests.

### 2.4.7.5  Water Penetration Test

***ASTM E 1105***[9] defines a method for testing curtain wall assemblies for water penetration. The test specifies a grid of water nozzles for applying a uniform spray to the exterior wall surface at the approximate rate of 200 l/m² per hour (equivalent to a rainfall intensity of 200 mm per hour – *Zwayer and Johnson*[33]). The test procedure specifies a spray duration of 15 minutes which is generally inadequate for MV/SS wall testing. More time is required to wet various wall components (such as the masonry or batt insulation) for leakage paths to develop. The test is typically run without any externally applied pressure (contrary to what is implied in the title for the test procedure).

As noted in Zwayer and Johnson[33], the 200 l/m² per hour rate of water application substantially exceeds most local rainfall conditions but is nevertheless appropriate because the water is applied in the absence of a pressure differential and the test must be capable of producing a leakage path in a reasonable time frame. It should also be noted that accumulative runoff down the face of a tall building can produce a similar rate of wetting. Brook[36] argues that when a number of qualifying assumptions are made with respect to lateral and upward water flow and the effect of building projections, the accumulative run–off would exceed the 200 l/m² per hour rate for building heights greater than 36 m.

A wall can be tested as one entity or as a collection of individual components. Masking in the form of 6 mil plastic sheeting can be taped on the exterior to protect components from water spray. Leaks observed during

the test can therefore not be attributed to the infiltration through a masked component. (*Kudder and Lies*[2]).

***Advantages*** of the test procedure include the ease with which different areas of the wall can be accessed. The spray rack can be easily moved around by swingstage or by ropes suspended from the roof.

***Disadvantages*** include the difficulty of locating a water supply that can deliver sufficient water pressure and volume.

Note that preliminary tests can be undertaken by flooding various components. Flooding can be undertaken with a simple garden hose or in a modification to AAMA 501.2[40] a fire hose can be used to apply large volumes of water at substantial pressure to a structure. This method is recommended in Nicastro[1] when a large area must be tested or when leakage is not anticipated (if it doesn't leak with a fire hose, it doesn't leak). It is difficult, however, to isolate a specific source of leakage with the fire hose method since multiple components are being sprayed. For small areas such as a suspected leaky window sill, peel and stick draft stop can be used to create dammed up areas to create locally severe conditions for water leakage testing.

### 2.4.7.6 Nozzle Tests *(from Zwayer and Johnson[33])*

***AAMA 501.2-83*** is a field test procedure to determine the resistance to water penetration of joints within a curtain wall system. Joints are tested, beginning with the lowest joint in the test area, utilizing a hand held spray nozzle with control valve and pressure gauge. Water flow is adjusted to maintain a pressure of 0.21 to 0.24 MPa at the nozzle. Joints are tested in 1500 mm long segments for 5 minutes and the interior side of the wall is monitored for leakage. The tests is useful for checking the watertight-ness of sealants at masonry control and expansion joints including the soft joints under shelf angles and for checking the interfaces with other building materials such as windows, louvered openings and doors.

***Advantages*** include ease and speed of setting up and running the test.

***Disadvantages*** include the inability to detect leakage in some cases be-cause other wall components are absorbing the water.

### 2.4.7.7 Leak Testing with Organic Vapour Detection

Where the source of leakage is remote from the observed leakage site, detection of vapour from water mixed with an organic solvent can assist in tracing water movement.

When a likely leak source is found, water containing no more than 10% alcohol can be added to the suspected leak site. If the organic vapours

are detected at the remote site, then there is evidence that the source of the leak has been located. This procedure should not be conducted in the vicinity of possible ignition sources.

Moderately priced hand-held organic vapour detectors are available. For more information refer to May and Vassiliades[41].

### 2.4.7.8 Discussion

It is recommended in Zwayer and Johnson[33] that testing begin with ASTM E 1105[39] spray tests with all finishes in place and perhaps with some portions of the wall masked with plastic sheeting. Once leakage paths have developed and correlated with previously recorded leakage areas then the removal of interior drywall and batt insulation and finally exterior drywall can be useful to help trace the source of leakage. Follow-up nozzle testing is used to check sealed joints. Further testing with the combination of water permeance testing and wall drainage tests allows a good understanding of the leakage associated with the masonry components of the wall assembly (including flashings).

Other investigators emphasize the importance of a systematic approach to the diagnosis of leakage problems.

Kudder and Lies[2] "Diagnosing Window and Curtain Wall Leaks", argue that three things must be understood before testing begins:
- the extent of the problem
- the design intent
- the as-built conditions

Although Kudder and Lies[2] is directed to window and curtain wall leaks, many of the suggested procedures are directly transferable to MV/SS walls.

Nicastro[1] "A Scientific Approach to Water Infiltration Studies" proposes a rigorous application of the scientific method to diagnosing wall leakage problems. No field testing should be undertaken until a hypothesis is formed as to the likely sources of infiltration. Refer to Nicastro[1] for more detail on the proposed method. Note that the application of the scientific method is appropriate for all forms of field studies – not just leakage investigations.

### 2.4.8 Water Vapour Transmission Measurement

Current test methods require the collection of samples and laboratory testing by ASTM E 96[42]. No field measurement techniques are available.

ASTM E 96 allows two different procedures. The cabinet conditions for both are

23° C and 50% RH. In the dry cup test, desiccant is placed inside a cup to maintain 0% RH and the cup is sealed on top with the test specimen. Vapour flows from the 50% cabinet environment through the specimen and into the cup. The wet cup test is similar except water is placed inside the cup to maintain 100% RH and water vapour flows from the cup to the 50% cabinet environment. In both case the cup is weighed after a period of time to determine the moisture gain or loss. Wet-cup values are typically higher. For more information, see Hutcheon and Handegord[26].

The laboratory findings should be interpreted with caution considering that the test method does not consider the effect of temperature, the test is difficult to run and the test conditions of 50% to 0% or 100% to 50% are not representative of usual field conditions.

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# Chapter 3 – Behaviour, Deficiencies and Consequences

In order to successfully diagnose wall problems, an understanding of the basic engineering and building science issues is required. This chapter presents a brief overview of these issues along with typical MV/SS deficiencies and their associated consequences.

The importance of an accurate diagnosis for a problem wall cannot be overemphasized. In its absence, remedial work is likely to be successful only to the extent that statistical chance favours the investigator. Unsuccessful remedial work can be a substantial waste of money and resources, may accelerate the deterioration of the wall, and may make future investigations more difficult.

The following section reviews the various elements of a MV/SS wall system under the following headings:

- Strength Behaviour
- Strength Deficiencies/Consequences

- Serviceability Behaviour
- Serviceability Deficiencies/Consequences

- Building Science Behaviour
- Building Science Deficiencies/Consequences

Each of these categories may include examples of major or minor distress as defined in economic terms and/or risk to public safety. There is also interaction between the categories where, for example, a building science problem such as condensation in the stud space might cause sufficient corrosion on steel parts that the strength of the steel stud backup is compromised. Distress is typically due to inadequacies in materials, design, construction or maintenance. Usually, some combination of the above factors are involved in any one investigation[1].

Note that the lists of deficiencies and consequences do not include every possible contingency. There are an infinite number of ways to design and construct a building the wrong way – only the more common deficiencies are addressed here. In addition, for a particular wall element, only those headings that apply are included.

Information can be located in this section by using the index in Table 3 –1 which directs the reader to the appropriate item discussed in the text. The table lists alphabetically:
- Symptoms observed, and
- Wall element under distress

The CMHC document, Exterior Wall Construction in High-Rise Buildings by Drysdale and Suter[1], and the CSSBI document, Lightweight Steel Framing Design Manual by Trestain[2], are the fundamental references for much of this chapter. Pertinent information from the CMHC and CSSBI documents has been summarized here along with information from a number of

other reference documents. Note that examples of good practice have not generally been provided because they are available elsewhere.[1,2,13]

| Wall Component, Behaviour or Symptoms/Diagnoses | Strength | Serviceability | Building Science |
|---|---|---|---|
| Air barrier | *See 3.9.1, 3.10.2, 3.10.3, 3.13.1* | *See 3.10.4, 3.16.3* | *See 3.1.5.1, 3.1.5.2, 3.5.4.1, 3.9.4, 3.10.5, 3.12.3, 3.12.4* |
| Air leakage | *See air barrier* | *See air barrier* | *See air barrier* |
| Air space | *See 3.8.1* | | *See 3.1.5.1, 3.5.4.1, 3.8.2* |
| Brick | *See 3.1.1, 3.3.1, 3.5.1.1, 3.5.2.1* | *See 3.4.1.1, 3.4.1.2, 3.4.1.3, 3.4.1.4, 3.4.2.1, 3.4.2.2, 3.5.3.1, 3.15.3, 3.16.3* | *See 3.1.5.1, 3.5.4.1* |
| Bridging | *See 3.13.1, 3.13.2, 3.14.1, 3.14.2, 3.21* | | |
| Caulking | *See sealants* | *See sealants* | *See sealants* |
| Cavity | *See air space* | *See air space* | *See air space* |
| Control joints | *See cracking* | *See cracking* | *See cracking* |
| Corrosion | *See 3.2.2, 3.6.2, 3.7.2, 3.15.2, 3.21* | *See 3.2.3, 3.3.3* | *See 3.1.5.2, 3.6.4, 3.9.4, 3.10.6* |
| Cracking – flexural | *See 3.1.1, 3.1.2, 3.3.2* | *See 3.1.1, 3.1.2, 3.1.3, 3.1.4, 3.3.3, 3.6.3* | *See 3.5.4.1* |
| Cracking – horizontal movement | | *See 3.2.2, 3.4.1, 3.4.2.1, 3.19.2, 3.20.1* | *See 3.5.4.1* |
| Cracking – vertical movement | | *See 3.2.3, 3.4.1, 3.4.2.2, 3.4.2.3, 3.20.1* | *See 3.5.4.1* |
| Deflection | | *See 3.1.3, 3.2.3, 3.3.3, 3.4.1.1* | |
| Drywall – exterior | *See sheathing - exterior* | *See sheathing - exterior* | *See sheathing - exterior* |
| Drywall – interior | *See sheathing - interior* | *See sheathing - interior* | *See sheathing - interior* |
| Drying | | | *See 3.1.5.2, 3.9.4* |
| Dust shadowing | | | *See 3.9.4* |
| Efflorescence | | | *See 3.5.4.1, 3.7.2, 3.10.6, 3.19.3* |

| TABLE 3 - 1 Index - Refer to Item Numbers Indicated | | | |
|---|---|---|---|
| **Wall Component, Behaviour or Symptoms/Diagnoses** | **Strength** | **Serviceability** | **Building Science** |
| Expansion joints | *See cracking* | *See cracking* | *See cracking* |
| Flashings | | | *See 3.5.4.1, 3.7.1, 3.7.2, 3.9.4, 3.19.3* |
| Freezing and Thawing | | | *See 3.5.4.2, 3.5.4.3, 3.7.2, 3.19.3* |
| Head track | *See window* | *See window* | *See window* |
| Insulation – batt | | | *See 3.1.5.2, 3.9.4, 3.10.6, 3.11.1, 3.11.2* |
| Insulation – exterior | | | *See 3.1.5.2, 3.11.2* |
| Lintels | *See 3.3.1, 3.3.2* | *See 3.3.3* | |
| Moisture | *See 3.21* | *See 3.1.4, 3.4.1.2, 3.4.1.4* | *See 3.1.5.1, 3.1.5.2, 3.5.4.1, 3.5.4.2, 3.5.4.3, 3.6.3, 3.6.4, 3.7.2, 3.8.2, 3.9.3, 3.9.4, 3.10.6, 3.19.3* |
| Moisture movements | | *See 3.4.1.2, 3.4.1.4, 3.4.1.5, 3.5.3.1* | |
| Mortar and mortar joints | *See 3.5.1.1, 3.6.2* | *See 3.5.3.1, 3.5.3.2* | *See 3.1.5.1, 3.5.4.1, 3.5.4.3, 3.6.4* |
| MV/SS walls as a system | *See 3.1.1, 3.1.2* | *See 3.1.3, 3.1.4, 3.15.3, 3.16.3* | *See 3.1.5.1* |
| Movement | *See cracking, top track* | *See cracking, top track* | *See cracking, top track* |
| Parapets | *See 3.19.1* | *See 3.4.2.2, 3.19.2* | *See 3.5.4.1, 3.19.3* |
| Rain screen | | | *See 3.1.5.1, 3.5.4.3, 3.8.2* |
| Sealants | | *See 3.16.3, 3.20.1* | *See 3.5.4.1* |
| Sheathing – exterior | *See 3.6.2, 3.9.1, 3.14.1, 3.14.2.1, 3.16.2* | *See 3.9.2* | *See 3.1.5.2, 3.6.4, 3.7.2, 3.9.3, 3.9.4, 3.10.6* |
| Sheathing – interior | *See 3.12.1, 3.14.1, 3.14.2.1, 3.16.2* | *See 3.12.2* | *See 3.10.6, 3.12.3, 3.12.4* |
| Shelf angles | *See 3.2.1, 3.2.2, 3.21* | *See 3.2.3, 3.4.2.2* | *See 3.2.4* |
| Sill track | *See window* | *See window* | *See window* |

| Wall Component, Behaviour or Symptoms/Diagnoses | Strength | Serviceability | Building Science |
|---|---|---|---|
| Spalling | | | *See 3.5.4.1, 3.5.4.2, 3.10.6, 3.19.3*<br>*See also freezing and thawing, efflorescence* |
| Studs – built-up at jamb | *See 3.17.1, 3.17.2, 3.21* | *See 3.17.3, 3.17.4* | *See 3.17.5* |
| Studs – typical | *See 3.1.1, 3.13.1, 3.13.2, 3.15.1, 3.16.1, 3.19.1, 3.21* | *See 3.9.2, 3.12.2, 3.13.3, 3.13.4, 3.15.3* | *See 3.1.5.2, 3.13.5* |
| Thermal bridging | | | *See 3.1.5.2, 3.2.4, 3.6.4, 3.9.3, 3.9.4, 3.10.6, 3.11.2, 3.16.4* |
| Thermal expansion and contraction | | *See 3.2.3, 3.4.1.3, 3.4.1.4* | |
| Thermal – R value | | | *See 3.1.5.2, 3.11.2* |
| Ties | *See 3.1.1, 3.6.1, 3.6.2, 3.8.1, 3.13.1, 3.21* | *See 3.1.3* | *See 3.6.4, 3.10.6* |
| Track – bottom | *See 3.9.4, 3.10.6, 3.13.2, 3.15.1, 3.15.2, 3.19.1, 3.21* | *See 3.15.3* | *See 3.15.4* |
| Track – top assembly | *See 3.16.1, 3.16.2, 3.21* | *See 3.10.4, 3.16.3* | *See 3.16.4* |
| Vapour retarder | *See 3.10.2, 3.10.3,* | *See 3.10.4* | *See 3.9.4, 3.10.5, 3.12.3, 3.12.4* |
| Vents and weepers | | | *See 3.1.5.1, 3.5.4.1, 3.5.4.3* |
| Weepers | *See vents* | *See vents* | *See vents* |
| Window – head and sill members | *See 3.18.1, 3.18.2, 3.21* | *See 3.18.3, 3.18.4* | *See 3.18.5* |

**TABLE 3 - 1**
*Index - Refer to Item Numbers Indicated*

## 3.1   MV/SS Walls as a System

### 3.1.1 MV/SS Walls as a System – Strength Behaviour   *(from Trestain/Rousseau[3] )*

The primary structural function of a MV/SS wall is to withstand the effects of wind and earthquake. No axial load other than self-weight should be placed on the brick veneer or the steel stud backup.

For a complete SS/BV wall analysis, the distribution of wind (and earthquake) forces should be considered both before and after flexural cracking of the brick veneer. Before cracking gives the maximum tie loads and after cracking the maximum load on the steel stud backup.

*Before Cracking:* With uncracked brick veneer, the distribution of internal stresses in the veneer, brick ties and steel stud backup is a highly indeterminate problem which is influenced by:

- The relative stiffness between the stud and the brick veneer
- The stiffness of the brick ties
- The top and bottom track stiffnesses
- The top of brick restraint
- Whether the wind loads the veneer, the backup or both
- The presence of openings such as windows
- The horizontal versus the vertical bending stiffness of the brick veneer (if boundary conditions permit two way bending of the veneer).

All these effects need to be taken into account for an accurate prediction of load distribution between the various elements in the wall system.*

Nevertheless, some useful understanding can be gained from an approximate analysis which also leads to a reasonable estimate of the maximum load on brick ties. (See Figure 3 – 1.) The following simplifying assumptions can be made:

- The stud backup and the uncracked brick veneer are separate simply supported flexural members each capable of carrying load.
- The brick veneer and the steel stud span vertically only.
- The brick and stud span lengths are equal.
- The brick and stud are constrained to deflect the same amount under wind load because they are connected by brick ties.
- The end supports for the brick and the studs do not move under wind load.

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\*     *The three dimensional MVSS finite element computer program included with this project has the capability of handling the variables outlined here and plus crack propagation in the brick veneer. See Chapter 4 and the MVSS User Manual.*
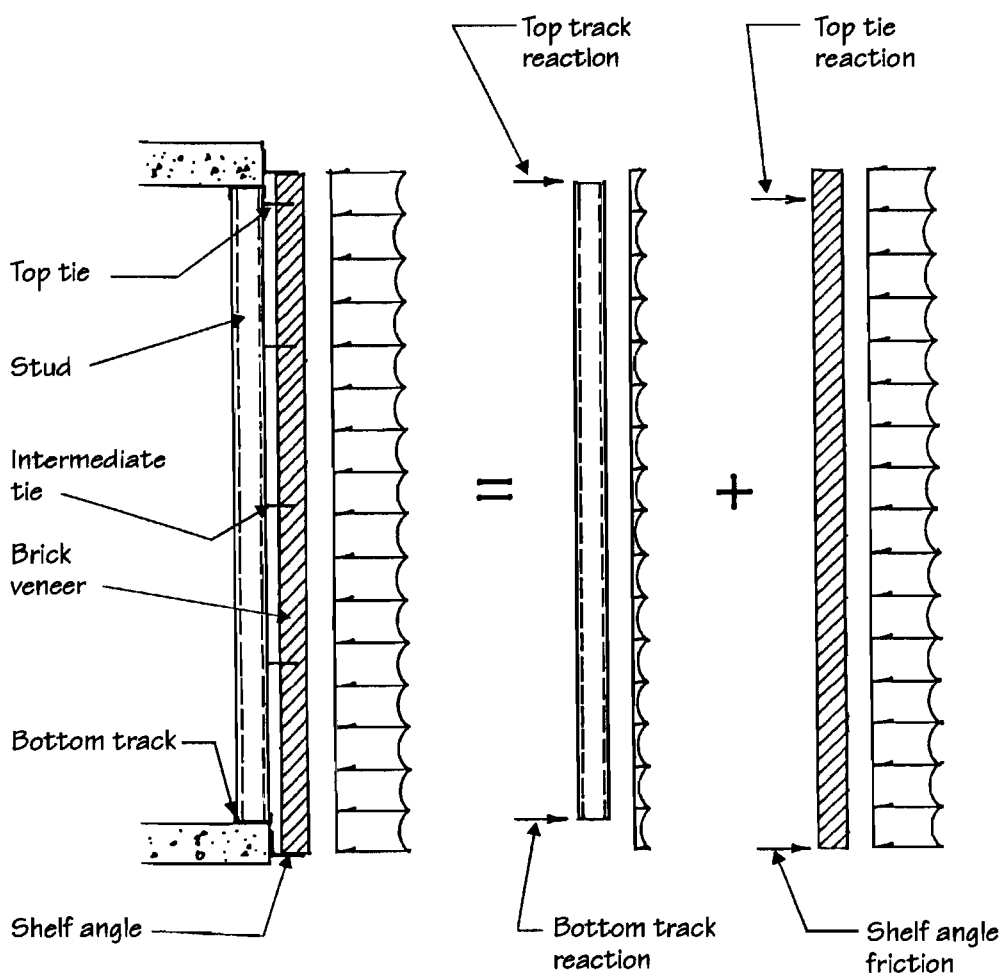
FIGURE 3 - 1        DISTRIBUTION OF LOAD BETWEEN UNCRACKED BRICK
                    AND STEEL STUD BACKUP

Based on these assumptions and by equating deflections, a load sharing formula can be derived.

$$W_{BRICK} = W_{TOTAL} / [1 + (EI)_{STUD} / (EI)_{BRICK}]$$

where:

$W_{STUD}$ and $W_{BRICK}$ = the wind load carried by the stud and brick respectively acting as simply supported beams

For 92 mm x 1.22 mm steel studs @ 400 mm o.c. and 90 mm brick veneer, it can be shown that the brick carries 88% of the wind load and the stud the remaining 12%.

*After Cracking:* After the brick veneer cracks, the brick is assumed to form a hinge at midspan and lose its ability to span from floor to floor. Testing and computer studies[4,5] have indicated that, in fact, the cracked brick retains a portion of its initial flexural strength and stiffness but this is typically ignored in design and the full wind load is applied to the steel stud backup. This is the design approach required by CSA S304.1 for new buildings.

*Tie Loads:* From the previous approximate analysis before veneer cracking, the top tie carries the maximum load of:

$$(0.88 \ W_{TOTAL}) \times 1/2 = 0.44 \ W_{TOTAL}$$

This agrees well with the results of an indeterminate analysis[4] with the wind load applied to the veneer only. It is also consistent with the tie design requirements in CSA S304.1[6] where for flexible backup systems such as steel stud, all ties are to be designed for 40% of the tributary lateral load on the stud (but not less than double the tributary lateral load on the tie).

The indeterminate analysis also indicates that after cracking, again with winds applied to the veneer, the load on the midheight tie nearest the crack will approach this same value. Since the location of the midheight crack cannot be accurately predicted the usual approach is to design all ties in accordance with the 40% rule. Obviously, the old notion of designing for a tributary area equal to the horizontal x the vertical tie spacing is inadequate.

*Veneer Cracking:* Both the approximate and the indeterminate analysis indicate that brick veneer cracking should be anticipated in the design of SS/BV wall systems. Continuing the approximate analysis, it can be shown that for a wall 2590 mm high with 1.2 kPa of total wind load, the actual flexural tensile stress in the brick is 0.66 MPa. From Drysdale and Breton[4], the ultimate value for the flexural tensile stress in brick can range from 0.2 – 0.9 MPa. Therefore, with an actual stress of 0.66 MPa, veneer cracking is likely but not certain. Note that the probability of veneer cracking increases as the height of the wall increases but is substantially reduced if boundary conditions for the veneer allow two way bending to develop.

### 3.1.2  MV/SS Walls as a System – Strength Deficiencies/Consequences

For deficiencies and consequences of failure of individual wall components, refer to the relevant item number.

Note that the formation of the first flexural crack of the brick veneer is not treated as a strength deficiency but rather as a serviceability limit state. See Item 3.1.1.

The formation of a second crack can be a concern structurally where the cracking spacing is less than the tie spacing and a mechanism can form[1] . See Figure 3-2.
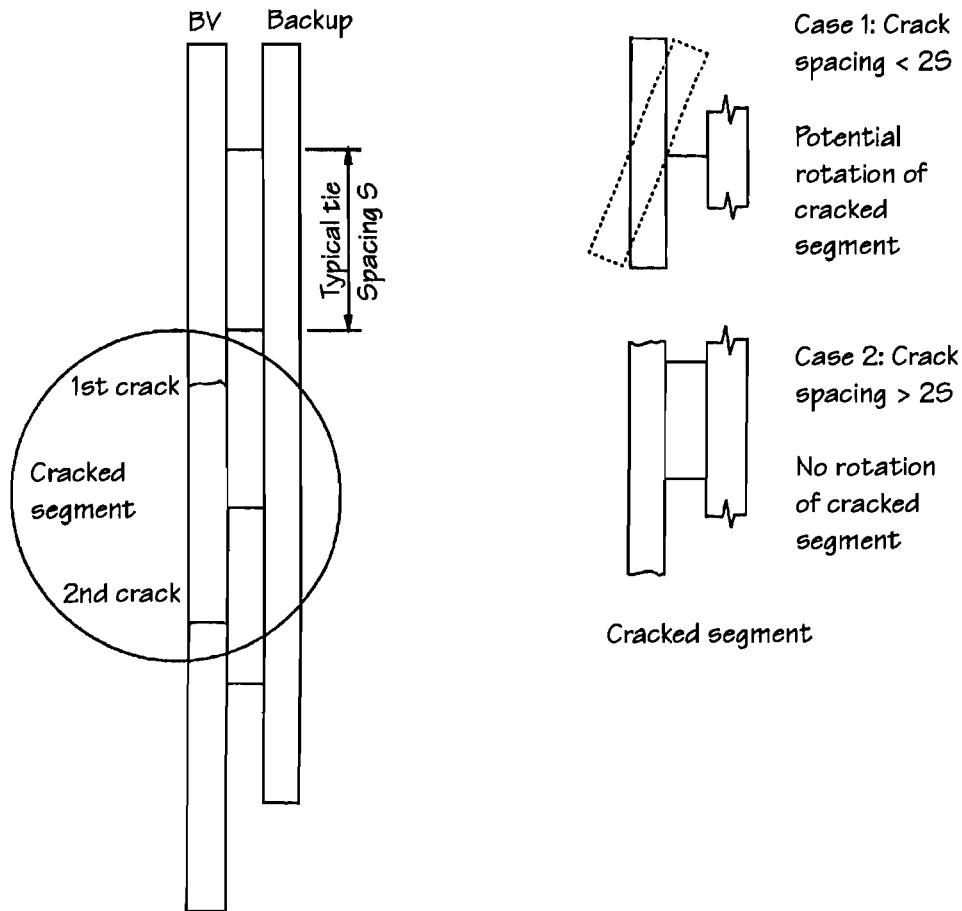


<table>
<tr><td>FIGURE 3 - 2</td><td>ILLUSTRATION OF POTENTIAL INSTABILITY OF CRACKED VENEER FOR LARGE TIE SPACING<br>(Reproduced from Drysdale and Suter [1])</td></tr>
</table>

### 3.1.3 MV/SS Walls as a System – Serviceability Behaviour

The steel stud backup is designed to have adequate stiffness to control the size of the first flexural crack, once formed. As discussed in Items 3.1.1 and 3.1.2, the formation of the first crack is a serviceability limit state not a strength limit state – see Figure 3-3.

CSA Standard S304.1[6] requires that the full specified load be applied to the backup and that the deflection of the veneer should not exceed the stud height divided by 600. The veneer deflection is the sum of the bending deflection of the steel stud plus the mechanical play of the ties plus the deformation of the ties under load. The deformation of the top and bottom track are excluded from the calculation. S304.1 also provides an alternative design method wherein the stud deflection is restricted to L/720 provided the deformation under load and the mechanical play of the ties is restricted.
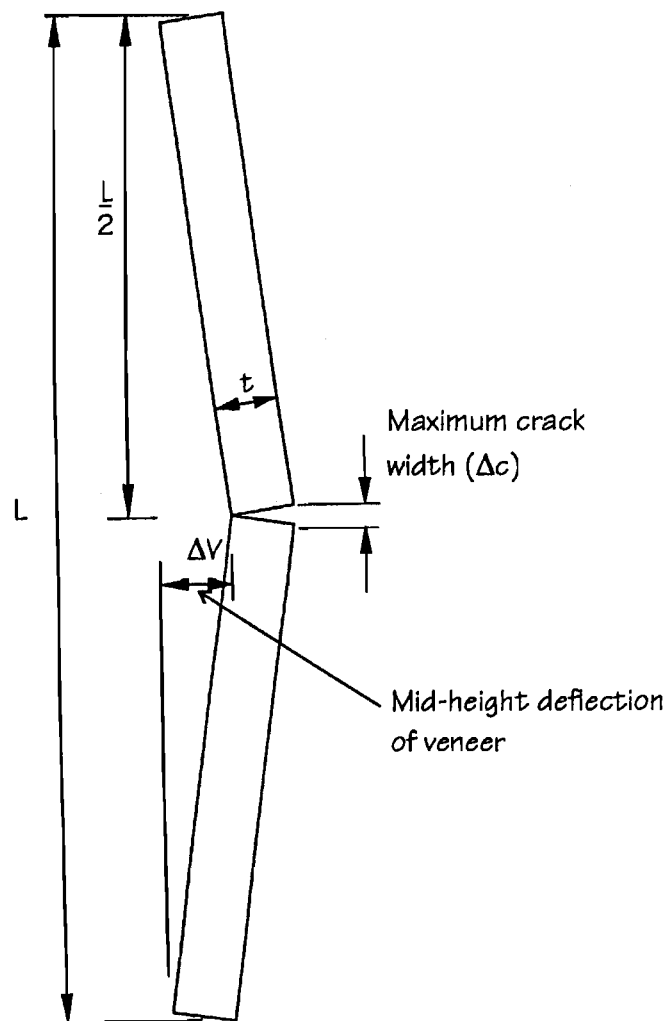


FIGURE 3 - 3                    SCHEMATIC OF VENEER CRACK WIDTH
                                (Reproduced from Drysdale and Suter[1] )

### 3.1.4 MV/SS Walls as a System – Serviceability Deficiencies/Consequence

As discussed in Item 3.1.1, flexural cracking in the brick veneer is not certain but is sufficiently probable that it should be treated as a design condition. The more flexible the steel stud backup the more likely the brick is to crack and the larger the crack will be once it is formed.

In full scale tests with the simultaneous application of wind and rain[5], there was no significant increase in the amount of rain water penetrating the brick veneer as a result of flexural cracking provided the cavity was fully pressurized. When the cavity was not pressurized, a significant increase in the amount of water penetrating the brick veneer occurred.

The effect of varying crack size was not a variable in the full scale testing project but it seems reasonable to conclude that a larger crack is likely to let in more water under conditions of no or partial cavity pressurization than a smaller crack. If the cavity is pressurized, then a larger crack size is likely of little consequence from a water penetration point of view.

### 3.1.5 MV/SS Walls as a System – Building Science Behaviour

#### 3.1.5.1 *Rain Screen*

MV/SS walls are designed as pressurized rain screen systems. For a detailed description, the reader is referred to a number of useful references[5, 7-13]. A brief description follows.

The open rain screen wall system is shown diagramatically in its simplest form in Figure 3–4.

It consists of an exterior rain screen, a cavity and an interior air barrier system. The exterior rain screen is vented to the outside such that changes in exterior air pressure are followed closely by changes in cavity air pressure. The air pressure between the cavity and the exterior is thus equalized and there should be little or no pressure drop to force rain through openings in the rain screen. The air pressure difference across the wall is carried instead by the interior air barrier assembly.

*Advantages*

• The exterior rain screen is not sensitive to imperfections. Any accidental openings (for example in sealants or mortar joints) are not likely to contribute to additional rain penetration since the pressure difference driving the water penetration is eliminated.

Exterior rain screen

Interior air barrier

Water

Vent opening

Exterior high pressure

Interior low pressure
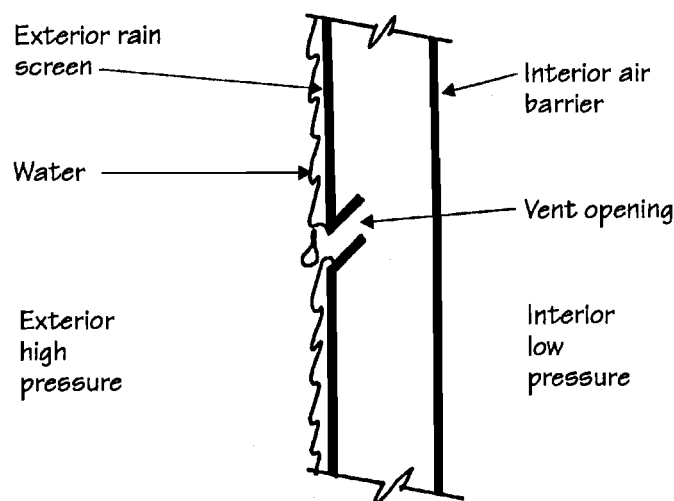
FIGURE 3 - 4    THE OPEN RAINSCREEN WALL SIMPLIFIED

- There is a second line of defence to water penetration. Water that passes through the exterior rain screen does not bridge the cavity but runs down the inside face of the rain screen to drain out.
- The air barrier system is protected from the deleterious effects of water, ultraviolet radiation and temperature extremes.
- Because the interior air barrier does not get wet, minor air leakage through it will not contribute to water penetration. (Leaky air barriers can, however, have other consequences. See Item 3.10.6.)
- Air circulation in the cavity can assist drying.

*Disadvantages*

- True pressure equalization requires careful design and construction. See the discussion that follows:

In order to achieve pressure equalization, a number of design and construction details require attention:

- The vents in the rain screen must have adequate area. The required vent size is a function of the volume of the cavity, the air barrier leakage rate, the flexibility of the air barrier assembly and the dynamic nature of wind gusts.

- The air barrier should have a low leakage rate. It is possible to have a pressure equalized wall in combination with an air barrier with a high leakage rate but this would require considerable air flow through the rain screen to supply the make-up air. While the pressure equalized principle would not be offended, water penetration through the rain screen could still occur with droplets transported along with the moving air through openings.

- Horizontal air flow in the cavity must be controlled. Horizontal air flow occurs because positive wind pressure on one wall is always accompanied by negative wind pressure on the adjacent side walls. See Figure 3-5. This horizontal air flow substantially defeats other efforts to create a pressure equalized wall[11]. Vertical baffles are required at least at the building corners. Some researchers argue for complete compartmentalization of the cavity with vertical baffles as frequent as 3 m o.c. Horizontally baffling provided by shelf angles at every floor level are normally considered adequate. Note that deliberate horizontal baffling may be required in some locations such as near the top of the building to isolate the wall cavity from the parapet.
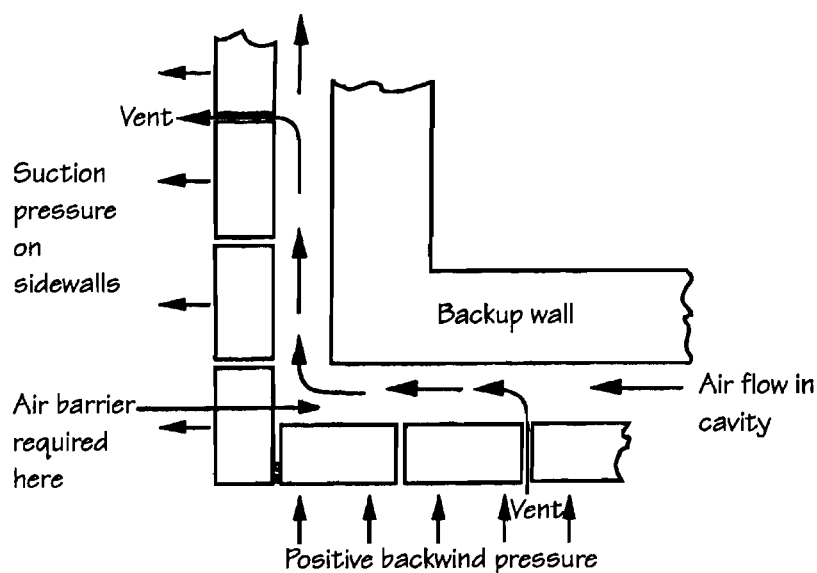


FIGURE 3 - 5        LACK OF PRESSURE EQUALIZATION DUE TO CAVITY HORIZONTAL AIR FLOW

Some building scientists have argued that a true open rain screen wall is not practical largely because pressure equalization is difficult to achieve. They have proposed another type of wall, designated the drain screen, which has similar construction details to the rain screen except that no particular effort is made to achieve pressure equalization. With this design approach, water will penetrate the exterior rain screen and efforts should be focused on insuring it does not bridge the cavity and can be drained out. Many current walls although designed as pressure equalized rain screens, are probably closer to the drain screen principle for a variety of reasons including leaky air barriers, inadequate vent area and the absence of vertical baffles (especially corner baffles) to inhibit horizontal air flow in the cavity.

In addition, brick veneer walls may leak water even in the absence of any pressure differential.

One possibility is gravity assisted flow through accidental openings that divert water inwards and downwards. These openings may be present due to construction errors or due to post-construction deterioration.

A similar but more pervasive mechanism has been reported by Newman and Whiteside[14]. In head joints, small downward sloping paths exist in cracks in the mortar to brick interface. These paths fill by gravity or capillary suction when the outside surface of the brick is wet. A hydrostatic driving force (potentially equal to the height of a course of brick – approximately 70 mm of water or 700 Pa) is available to drive water into the cavity. They found experimental support for this hypothesis by applying a back pressure to the veneer up to the point when leakage stopped. The required back pressure to eliminate leaks varied from 25 mm to 40 mm of water representing good and bad construction respectively. These hydrostatic heads are equivalent to driving forces of 250 to 400 Pa which in turn are equivalent to a significant portion of the design wind pressure (usually in the order of 1000 Pa – 1500 Pa)

The conclusion is that brick veneer walls leak and they leak more in the presence of a wind pressure differential.

See Figure B-1, Appendix B for a typical MV/SS detail at the floor level which illustrates the weep holes, vents, shelf angle, water barrier and flashings all of which are fundamentally important to the successful rain screen wall system.

### 3.1.5.2    *The Interaction Between Air/Vapour Flow and Thermal Performance*

MV/SS wall systems are typically built with batt insulation in the stud space and some form of sheathing on the outside face of the studs that may or may not include rigid insulation.

Drysdale and Kluge[9] studied these types of MV/SS walls in simulated winter conditions (-17° C minimum) with 35 - 40% relative humidity on the warm side (21° C) and a continuous pressure differential (75 Pa) across the wall . They included deliberate imperfections in the air barrier so that the vulnerability of the system to air leakage could be studied. Without exterior insulation they found that both the studs and the inside face of the exterior sheathing were subject to moisture accumulation. With 1" of rigid polystyrene insulation there was no moisture accumulation on either the studs or the inside face of the exterior sheathing. With 1" of rigid polystyrene insulation and with higher relative humidity (50 - 55%) on the warm side, condensation was observed on the inside face of the exterior sheathing. Refer to the research report for more detail.

Without exterior insulation, moisture accumulation and corrosion of the steel parts is very probable. In addition, the quantity of moisture accumulating on the exterior sheathing may be excessive - beyond the wetting capability of the sheathing and beyond the drying capability of the wall.

With exterior insulation (1" minimum) condensation on the steel parts can usually be ignored. In this case, the thermal bridging that occurs at stud locations is a virtue since they pump heat to the cold side and keep themselves above the dew point temperature.

The potential for condensation between studs can be studied using the classical resistance formula to determine temperature at any point in the wall for comparison with the calculated dew point. The resistance formula can be presented as follows:

$$t_x = t_i - (R_x / R_t)(t_i - t_o)$$

where:

$t_x$  =   the temperature at any point in the wall
$R_x$  =   resistance from the indoor air to any point in the wall at which the temperature is to be determined.
$R_t$  =   overall wall resistance from indoor air to outdoor air
$t_i$  =   indoor air temperature
$t_o$  =   outdoor air temperature

The calculation of the overall wall resistance, $R_t$, usually excludes the cavity air space and the brick veneer because they are "short-circuited" thermally by circulating air through weepers and vents. Keller[15] found that under some conditions the veneer and the air space might be included in the calculation of $R_t$.

### 3.1.6  MV/SS Walls as a System – Building Science Deficiencies/Consequences

For deficiencies and consequences of failure of individual wall components, refer to the relevant item number. See also Item 3.21, Corrosion.

## 3.2  Shelf Angles

### 3.2.1  Shelf Angles – Strength Behaviour

As structural elements, shelf angles are required to carry the weight of the brick veneer above and to transfer this weight to discrete connections along the edge of the floor slab.

A common design procedure is to size the horizontal leg (and by default the vertical leg) of the shelf angle for the cantilever bending moment illustrated in Figure 3–6.
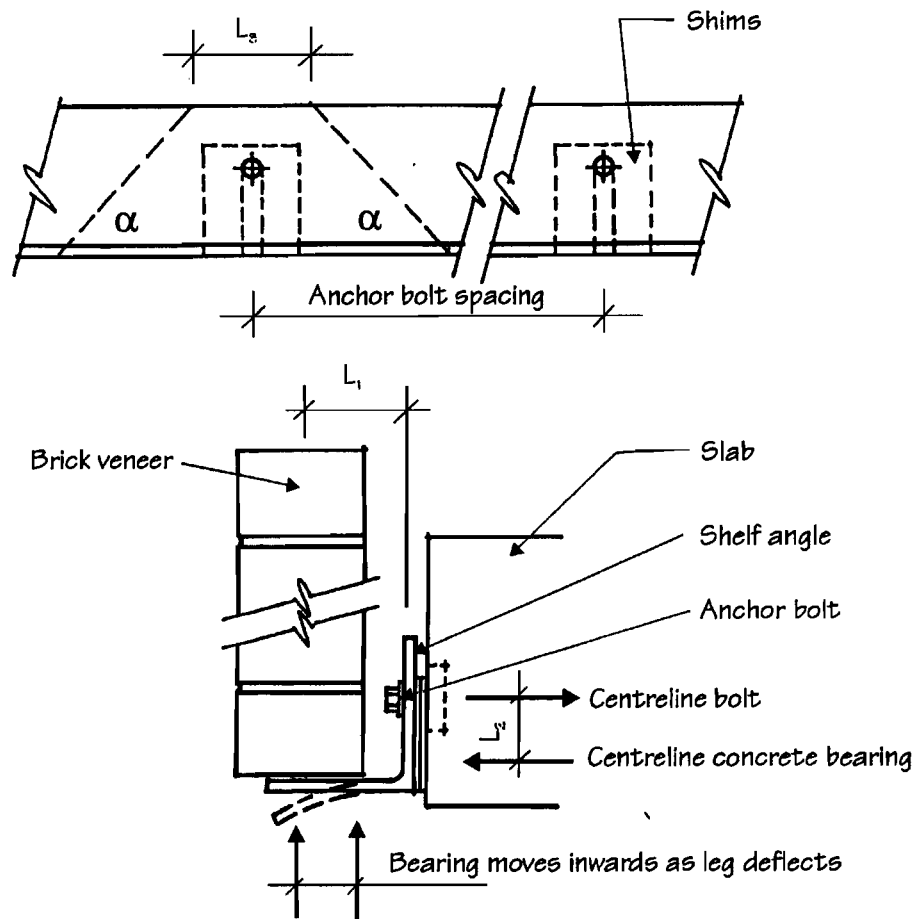
FIGURE 3 - 6          SHELF ANGLE AND ANCHOR BOLT DESIGN

The moment arm, $L_1$ , is taken from the centreline of the vertical leg to the centreline of the brick and the entire length of the angle between fasteners is assumed to be effective. The required steel thickness is based on the elastic section modulus. The concrete anchor is sized for combined shear and tension with the tension based on the lever arm, $L_2$ from centreline of bolt to the centroid of the concrete bearing area. Fastener spacings are typically limited to 1 m - 1.2 m. See Drysdale at al.[16] for a suggested procedure based on these assumptions.

The real behaviour of shelf angles is considerably more complex. Between connectors, the angle is subject to direct bending, torsion and the cantilever deflection of the horizontal leg. The load on the span of the cantilever is alleviated by arching action (see Loose Angle Lintels) and the bearing on the horizontal leg moves

inward as the leg deflects (Fig 3–6). At the location of fasteners, the entire weight of the veneer acts and the vertical leg of the angle is subject to locally high stresses and deformations. In the absence of more sophisticated analysis such as finite elements, Grimm[17] or Tide and Krogstad[18], an appropriate length of shelf angle defined by $L_3$ and $\alpha$ should be considered when calculating the resisting moment to cantilever bending of the leg. Note that a longer length of shelf angle is mobilized if plastification is permitted.

### 3.2.2 Shelf Angles – Strength Deficiencies/Consequences

Inadequate angle strength will result in yielding and excessive deflections that may compromise the movement joint beneath the shelf angle (See Vertical Differential Movement Item 3.4.2.2). Similar consequences result from inadequate anchorage details including:
* Missing anchors and/or bolts
* Bolts with nuts inadequately tightened
* Bolts fastened through oversized or burned holes in the vertical leg of the angle such that there is no bearing without considerable vertical movement of the angle
* Round washers used for shims which allow excessive rotation at the anchor location because lever arm $L_2$ (Fig. 3–6) is small.

See also Item 3.21 – Corrosion.

### 3.2.3 Shelf Angles – Serviceability Deficiencies/Consequences

Excessively flexible shelf angle details may compromise the movement joint beneath the shelf angle (See Vertical Differential Movements, Item 3.4.2.2).

If the shelf angle is unprotected and subject to periodic wetting, corrosion will develop. Corrosion products occupy more space and exert pressure at the steel masonry interface causing the supported masonry to move up or the shelf angle to move down. In either case, the horizontal movement joint capability is further compromised. See Grimm[17] for more detail. Corrosion stains may be evident on the face of the brick.

Thermal expansion in long uninterrupted sections of shelf angle may cause excessive longitudinal differential movements between the angle and the masonry. Maximum lengths of angle of 3.6 m with 6 mm open butt joints are recommended for design[17].

### 3.2.4 Shelf Angles – Building Science Deficiencies/Consequences

The shelf angle can be a source of thermal bridging particularly when a continuous bridge is formed via the top track detail. See Figure 3–7. Even without this

bridge with the top track, significant thermal bridging can occur through the floor slab where the shelf angle is not protected with insulation.
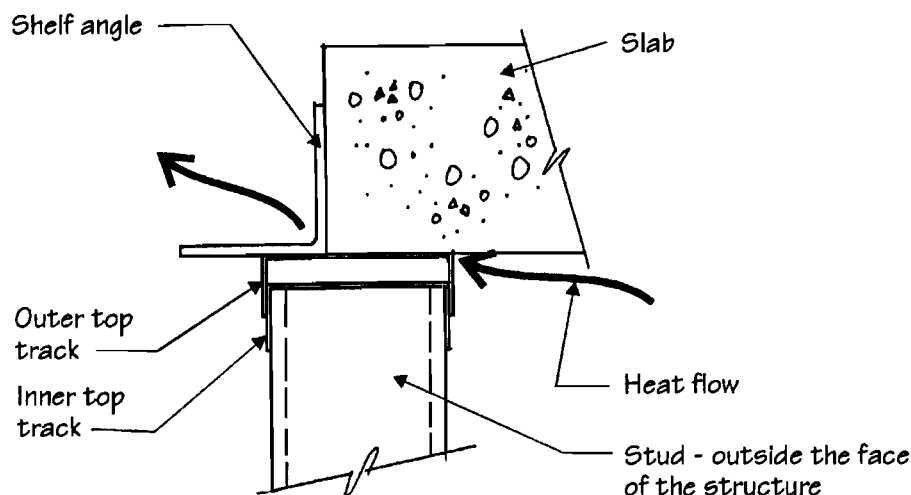


Note: In condominium type construction, it is common
     to provide shelf angles flush with the underside
     of the slab as illustrated here.

FIGURE 3 - 7      SHELF ANGLE - TOP TRACK THERMAL BRIDGE

## 3.3  Loose Angle Lintels

### 3.3.1  Loose Angle Lintels – Strength Behaviour

Loose angle lintels are typically used over smaller openings in the masonry and are designed as simply supported beams spanning from centreline of bearing to centreline of bearing. Usually, for design, the span length is limited to 2 m ± and only simple bending is considered. Arching is generally considered such that the weight of masonry carried encloses a 45° triangle. See Fig. 3–8. The length of bearing must be sufficient to keep brick bearing stresses under allowable limits and to insure that the lintel does not disengage during an earthquake.

The real behaviour of loose angle lintels is more complex than the simple bending

proposed above. Between end supports, the angle is also subject to torsion and the cantilever deflection of the horizontal leg.

For longer spans, the lintel is supported back to the structure with hot–rolled steel hangers at intermediate points.
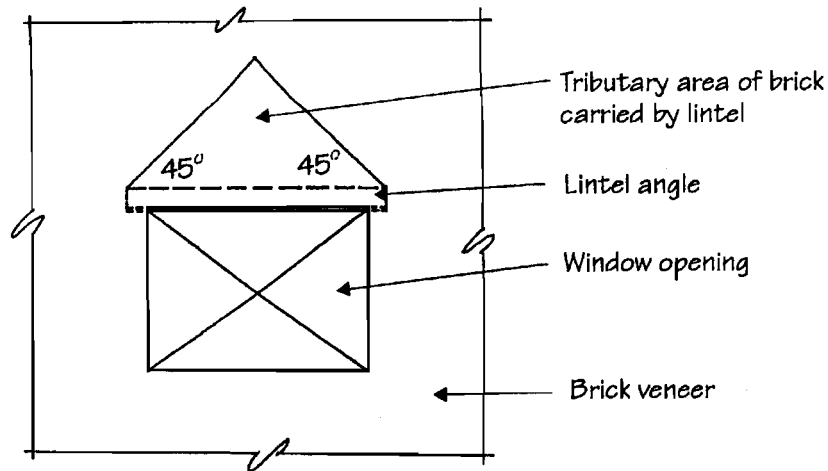


FIGURE 3 - 8          TRIBUTARY AREA OF BRICK CARRIED
                      BY LOOSE ANGLE LINTELS

## 3.3.2  Loose Angle Lintels – Strength Deficiencies/Consequences

Inadequately sized lintels will yield and deflect excessively and may lead to vertical cracks at the ends of the lintel or loading of the window frame.

Insufficient bearing length may result in a bearing failure (local crushing) of the brick under the shelf angle ends.

See also Item 3.21 – corrosion

## 3.3.3  Loose Angle Lintels – Serviceability Deficiencies/Consequences

Inadequate stiffness in the lintel may lead to vertical cracks at the ends of the lintel or loading of the window frame.

Volume increases associated with corrosion may cause masonry spalling/cracking at the end bearing locations. See Grimm[17] for more detail on volume expansion associated with corrosion.

## 3.4  Brick Veneer Horizontal and Vertical Movement Joints

### 3.4.1  Brick Veneer Horizontal and Vertical Movement Joints – Serviceability Behaviour

Relative movement between building components is the result of elastic and long-term deformation under load, moisture and thermal movements.

Movement can be restrained or unrestrained. Restrained movement, partially or completely restrained, results in internal stresses in the masonry and potentially large reactions on the restraining elements. Insignificant internal stresses or reactions result from unrestrained movement.

#### 3.4.1.1  *Elastic and Long-term Deflection Under Load*

This type of movement is important in the shortening that occurs in reinforced concrete structural frames under load and the deflection that occurs in reinforced concrete slabs and beams. Hot-rolled steel structural members are less of a concern with smaller deflections in the elastic range and virtually no additional long-term deflection.

From Drysdale and Suter[1], concrete (20 MPa) loaded to 8 MPa in compression will shorten elastically 0.4 mm/m with additional long-term creep equal to 1.5 – 2 times the initial elastic deformation.

Others[19,20] have reported that for reinforced concrete flexural members additional long-term deflection can range from 2 – 8 times the initial elastic deformation. Note that cantilevering slab systems are particularly susceptible to deflection problems.

#### 3.4.1.2  *Moisture Movements*

Concrete shrinks in a normal air environment with values ranging typically from .04 to .06 mm/m and in some cases to .10 mm/m. Most shrinkage occurs in the first two months after concrete is placed but continues indefinitely at decreasing amounts[21].

Concrete masonry units (concrete block – may not apply to concrete brick) shrinkage varies from .3 to .6 mm/m[1,21].

When removed from the kiln after firing, clay bricks absorb moisture as they come into equilibrium with the moisture content of air and begin to

permanently increase in size. The range of moisture expansion is not well defined for Canadian bricks but 0.2 mm/m is considered an average value[1,21]. The rate of moisture expansion is not uniform – 40% of the total expansion occurs in 3 months, 50% in 1 year and 100% in 60 – 100 years. The differential permanent expansion that occurs on the building depends on when the bricks were laid relative to the time of manufacture.

### 3.4.1.3  *Thermal Movements (from Drysdale and Suter[1])*

The thermal expansion or contraction of the brick veneer is taken from the temperature at the time of construction to the maximum or minimum temperature.

Recommended coefficients of thermal expansion are as follows:[1]

| | |
|---|---|
| Steel | .012 mm/m/°C |
| Normal Density Concrete | .011 mm/m/°C |
| Clay Brick | .006 mm/m/°C |

Note that solar heating (sol–air temperature) can increase the temperature range considerably say to 65°C in the summer for dark–coloured brick from winter time temperatures of say –25°C.

### 3.4.1.4  *Horizontal Movement in Clay Brick Veneer (from Drysdale and Suter[1])*

Total horizontal movement can be estimated from a formula:[1]

$$\Delta L = [0.2 + .006(T_e - T_r)]L$$

where:

$\Delta L$ = total contraction or expansion (–ve number indicates contraction), mm

$T_e$ = maximum or minimum temperature, °C

$T_r$ = reference temperature of the veneer (at the time of construction), °C

$L$ = length of the veneer, m.

0.2 = clay brick moisture expansion coefficient

0.006 = clay brick thermal coefficient

Note that moisture and temperature movements can be additive or can cancel out.

Examples with $T_e$ = 65°C to –25°C, L = 10 m

a) With $T_r$ = 40°C, max. expansion = 4 mm, max. contraction = 2 mm

b) With $T_r$ = 5°C, max. expansion = 6 mm, max. contraction = 0 mm

Note in a) if the brick is sufficiently aged at the time of construction and no moisture expansion occurs, then maximum contraction = 4 mm.

Refer to Drysdale and Suter[1] BIA Tech Notes[21-23] vertical movement joint recommendations for new construction.

### 3.4.1.5 *Total Vertical Differential Movement*

Differential vertical movement between the veneer and the building frame occurs primarily because the frame shortens due to load and shrinkage and the veneer expands due to moisture absorption aggravated by thermal expansion. See also Items 3.4.1.1, 3.4.1.2, 3.4.1.3. The differential movement is accommodated by movement (soft) joints under the shelf angles that typically occur at every floor level (but not always).

Drysdale and Suter[1] suggest that a 10 – 15 mm movement joint is typically adequate assuming a sealant with a cyclic deformability of ±25%. Note that the visible joint in the brick equals the movement joint plus the thickness of the shelf angle. See also Fenton and Suter[24], Grimm[17] and BIA Tech Note 18[21] for additional information.

## 3.4.2 Brick Veneer Horizontal and Vertical Movement Joints – Serviceability Deficiencies/Consequences

### 3.4.2.1 Horizontal Movements (Inadequate vertical expansion joints)

See Figure 3 – 9, 3 – 10 and 3 – 11. For recommended good practice with respect to vertical movement joint location and detailing, see Drysdale and Suter[1].

Type E

Type C

Type B

Type D — Type D

TOP FLOOR ELEVATION

Type E — Type D

Type B

Type D — Type D

Type A

BOTTOM FLOOR ELEVATION

Alternate locations of Crack E

PLAN VIEW
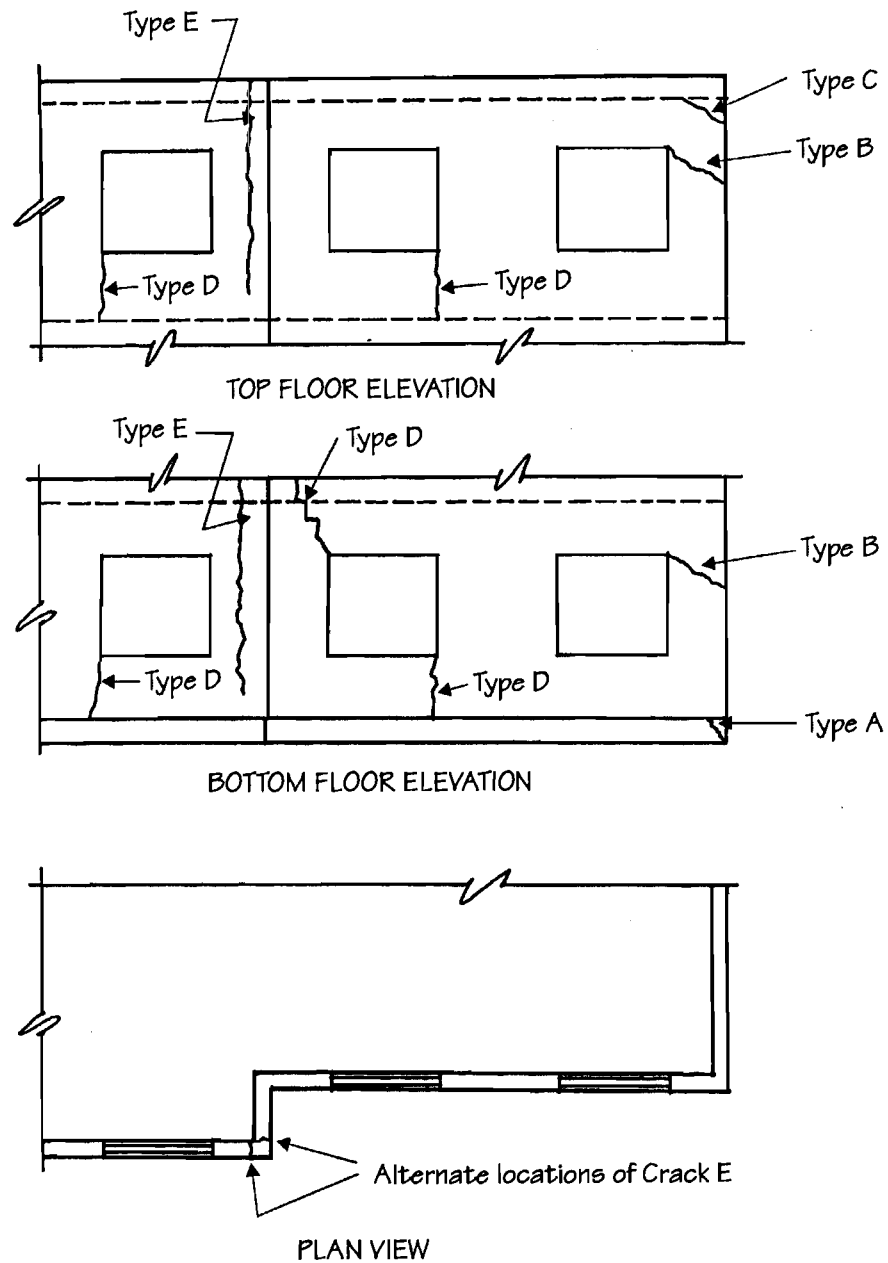
Note: See Figures 3 - 10 and 3 - 11 for details on Crack Types A - E

FIGURE 3 - 9       BRICK VENEER CRACK PATTERNS DUE TO
HORIZONTAL MOVEMENTS
(from Drysdale and Suter [1])

Relatively higher horizontal
expansion of veneer

Crack Type A in
corner of concrete
foundation wall

Concrete
foundation

Slip

CRACK TYPE A

Relatively free expansion at
top of veneer

Crack B in weak section due
to proximity to corner

Crack B in weak section
between closely spaced windows

Expansion restrained at
bottom by veneer support

CRACK TYPE B

Parapet expands more than veneer
below because of greater exposure.
Intersecting parapet will be out of
straight near this corner.

Roof slab

Crack C

CRACK TYPE C

FIGURE 3 - 10       BRICK VENEER CRACK PATTERNS DUE TO
HORIZONTAL MOVEMENTS
(from Drysdale and Suter[1])

Crack D - tension failure along weakest path -
usually mortar joints - sometimes units.

Restained thermal contraction
resulting in tensile masonry
stresses.

Crack D

CRACK TYPE D

Movement

CRACK TYPE E

Movement

Note: Crack Type E is typically
along the line of the first head
joint
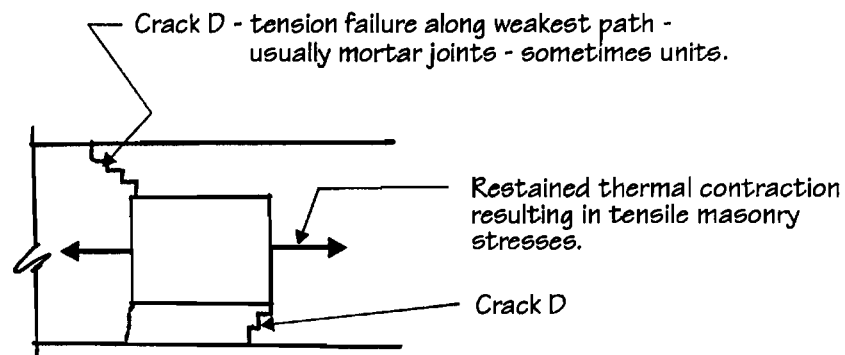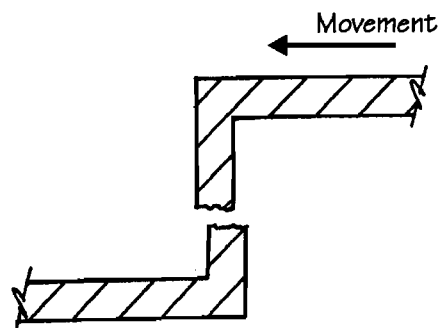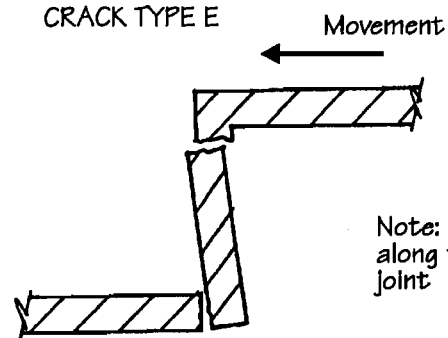
ALTERNATE CRACK TYPE E

FIGURE 3 - 11     BRICK VENEER CRACK PATTERNS DUE TO
HORIZONTAL MOVEMENTS
(from Drysdale and Suter[1])

### 3.4.2.2 Vertical Differential Movements (Inadequate horizontal expansion joints)

The absence of movement joints under shelf angles may result in large movements between the veneer and the structure at the roof level and high compressive stresses in the veneer at the lower stories.

Movement at the roof level can disturb parapets and damage roof flashings.

High compressive stresses at lower levels can lead to column buckling of the veneer, bulging of the veneer at shelf angles or local crushing of the veneer at shelf angles. Fenton and Suter[24] proposed three different veneer behaviours depending on the amount of masonry overhang at the shelf angle. Refer to Figure 3 – 12.

In Condition A with a small overhang, the masonry compressive stresses are concentrated in the small mortar joint at the toe of the angle and spalling of the veneer may result.

In Condition C with a large overhang, excessive rotation of the shelf angle may result especially if poor shimming details have been used (see Item 3.2.2). This shelf angle rotation may result in local crushing on the inside face of the veneer or lateral thrust on the veneer which in turn may result in bulging at the shelf angle or overall buckling of the veneer. Poor ties and/or and excessively flexible steel stud backup could aggravate this condition.

Condition B with a normal overhang, was considered the most stable where stress concentrations in the mortar are not high enough for spalling but the mortar may crush in the joint providing a partial movement joint.

See also Suter and Keller[25] for a historic review of the omission of horizontal movement joints by designers and contractors.

See also Drysdale[26] for the frequency of horizontal movement joint specification found in a in a survey of multi-family residential buildings.

CONDITION A – SMALL OVERHANG

CONDITION B– NORMAL OVERHANG

CONDITION C-LARGE OVERHANG
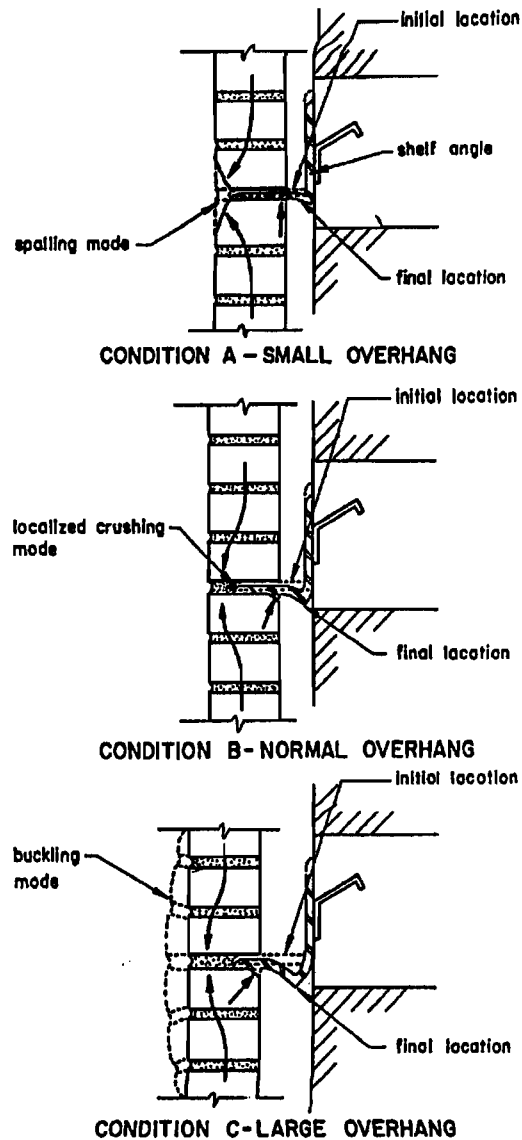
FIGURE 3 - 12        VENEER FAILURE CONDITION ASSOCIATED WITH
                     OVERHANG ON SHELF ANGLE
                     (Reproduced from Fenton and Suter[24] )

### 3.4.2.3 Vertical Differential Movements (Discontinuous shelf angle)

Projects with a horizontal movement joint under the shelf angle may suffer distress at locations where the shelf angle is discontinuous. See Figure 3 – 13.

Extent of shelf angle with
soft joint under.

Solid pier of masonry over
the height of the building.

CORNER - PLAN VIEW

Movement

Movement

Masonry shear crack
at end of shelf angle

CORNER - ELEVATION (ISOMETRIC)
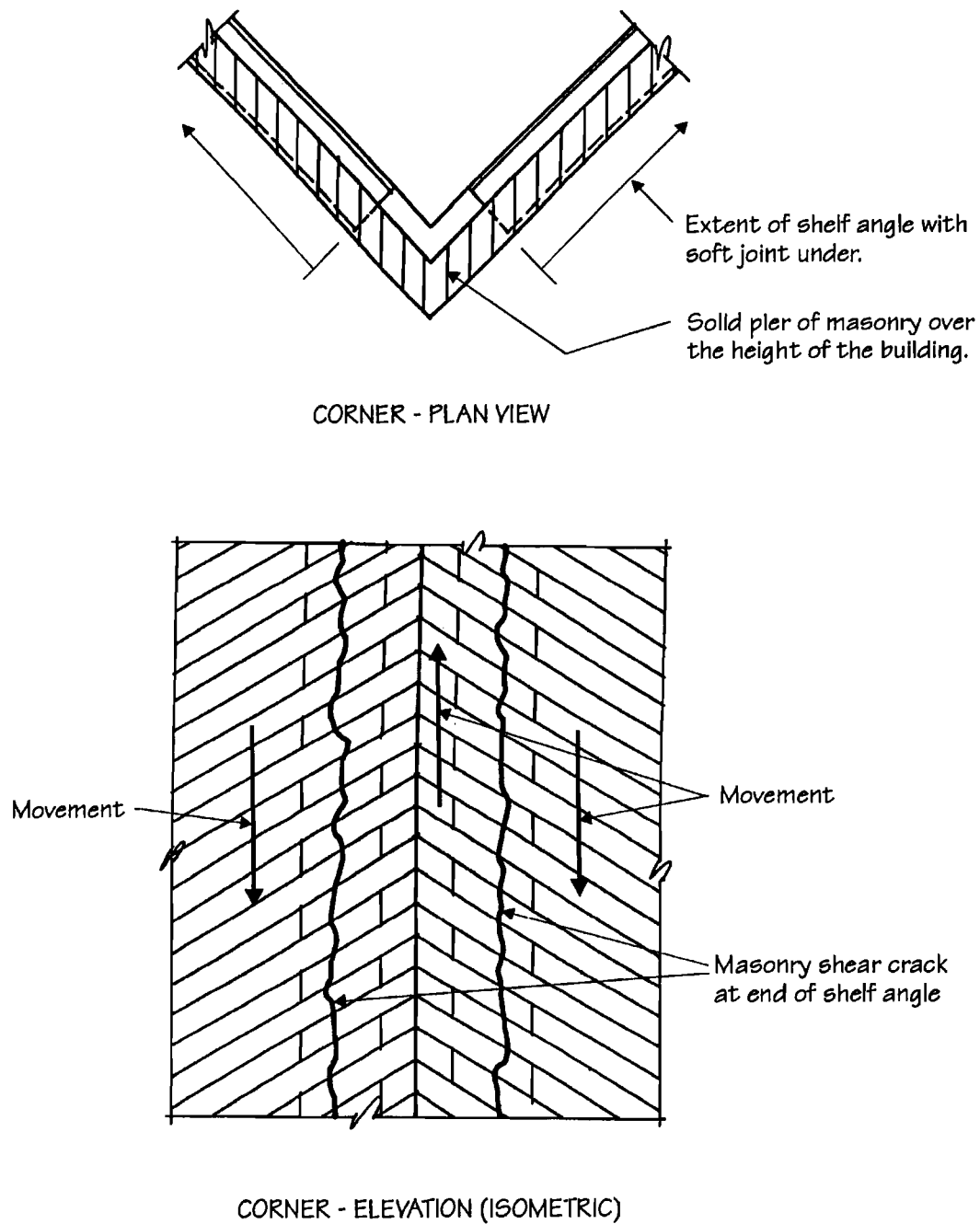
FIGURE 3 - 13     VERTICAL CORNER CRACKING ASSOCIATED
                  WITH DISCONTINUOUS SHELF ANGLE

## 3.5  Brick and Mortar

### 3.5.1  Brick and Mortar – Strength Behaviour

#### 3.5.1.1  *Flexural Strength*

For conventionally designed MV/SS wall systems, all the wind/earthquake load is assumed to be carried by the steel stud backup. For this design approach, only minimal structural strength is required in the brick veneer.

The veneer is required to do significant structural work, however, when brick ties and/or the steel stud backup are ineffective. The veneer is required to span vertically between missing ties or horizontally (where boundary conditions permit) to carry load the backup cannot. These conditions can be studied with the MVSS Finite Element computer program that is included as part of this project. Refer to Chapter 4.

### 3.5.2  Brick and Mortar – Strength Deficiencies/Consequences

#### 3.5.2.1  *Overhang*

A371-94[27] limits brick overhang to 30 mm or 1/3 of the width of the wythe whichever is less. Greater overhangs are potentially dangerous and require engineering review with respect to seismic performance, bearing stresses and eccentricity of the support stresses.

### 3.5.3  Brick and Mortar – Serviceability Deficiencies/Consequences

#### 3.5.3.1  *Cracking*

There are a number of possible sources for brick veneer cracking:

- No bond due to poor construction practices in hot or cold weather[28, 29]
- No bond due to poor materials – hard, low suction masonry units and/or high cement content mortars
- Excessively large flexural cracking due to inadequate backup stiffness/strength, missing or inadequate ties (See Item 3.1)
- Expansion/contraction cracking due to absence of vertical or horizontal control joints (or joints filled with mortar) (See Item 3.4)

Cracking in the brick veneer is a potential path for moisture to enter the cavity. The amount of water entering the wall is a function of crack size, the presence of water and the degree of pressure equalization. See Item

3.1.4 – MV/SS Walls as a System – Serviceability Deficiencies/Consequences.

### 3.5.3.2 *Mortar Joints*

Extruded, raked or struck mortar joints can be a source of increased water penetration since they are typically not well compacted against the masonry units and the time of construction and they are inclined to catch water running down the face of the wall. See Drysdale and Suter[1] for examples of mortar joint types.

### 3.5.4 Brick and Mortar – Building Science Deficiencies/Consequences

#### 3.5.4.1 *Efflorescence*

Moisture in the masonry dissolves salts that are present in brick and mortar. When this moisture migrates to the exterior and evaporates, the salts crystallize and leave a whitish (usually whitish – greenish and brownish stains occur occasionally depending on the salts present in the masonry) deposit on the face of the brick.

These deposits are usually seen as an aesthetic problem but under certain conditions salts can be deposited below the surface of the brick in the pores. When this occurs, the force of crystallization may cause spalling or disintegration of the brick surface. Brick with surface coatings are particularly susceptible to pore crystallization.

Efflorescence, while a problem, can be a useful diagnostic tool since it indicates excessive moisture in the brick. Typical sources include:

- Missing or inadequate flashing, damp–proofing, clearance from grade (see Item 3.7)
- Inadequate air barrier aggravated by excessive humidity inside the building (see Item 3.10)
- Poorly detailed or plugged/missing weepers and/or vents (see Item 3.5.4.3 – Vents and Weepers)
- Inadequate caulking at windows, shelf angles, control joints (see Item 3.20)
- Inadequate capping parapets and tops of walls exposed to the weather (see Item 3.19)
- Inadequate partitioning of air space (for example vertical currents of moist air being carried into parapet where cooled and moisture condenses out)
- Roof leak
- Cracking
- No drip shelf angle flashing allowing water from weepers above to flow into frogs of brick below.

Refer to Drysdale and Suter[1], BIA Tech Notes[30,31] and CBD 2[32] for more information.

Note that efflorescence frequently appears on new buildings, "new building bloom" and subsequently disappears with rain washings.

### 3.5.4.2 *Freezing and Thawing*

Bricks with inadequate freeze-thaw resistance are inclined to surface spalling. (See Item 3.5.4.3 Vents and Weepers for discussion on ice lens formation.)

Whether or not a brick fails in freeze-thaw depends on pore structure, rate of freezing, degree of saturation and thickness of the material. Saturation near the limit of the brick adsorption capacity is required. Standard criteria for freeze-thaw resistance (strength adsorption, saturation coefficient) are somewhat unreliable predictors[1]. Refer to Drysdale and Suter[1] and Suter and Maurenbrecher[33] for more information. Refer to Item 3.5.4.1, Efflorescence for moisture sources.

Keller[15] found almost continuous saturated conditions in combination with freeze-thaw temperatures on the inside surface of brick veneer during winter conditions. Concern was expressed that this could result in freeze-thaw spalling (for susceptible brick) that would be hidden and could compromise the structure integrity of the brick ties.

### 3.5.4.3 *Vents and Weepers*

Weepers that are missing, too infrequent or plugged (usually by mortar droppings) will compromise the ability of the rain screen to drain water that penetrates the brick veneer. Consequences include water build-up on the flashing and possible penetration to the building interior, shelf angle corrosion through flashing leaks, or the formation of ice lenses behind the brick. These ice lenses can exert sufficient pressure to displace units of brick outward.

The area of weepers and vents combined is required for pressure equalization of the rain screen. Drysdale and Wilson[5] stated that standard size and spacing of weepers and vents are adequate for pressure equalization.* This conclusion was based on typical vent and weeper detailing with open head joints at 800 mm o.c. Note that A371[27] requires only 70 mm² weeper holes at 800 mm o.c. which is approximately 1/10th the area of an open head joint. Vent size and spacing is designers choice but

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\*    *More current research suggests that only partial pressure equalization can be expected because of the short-term duration of wind gusts.*

frequently matches the weepers.

The need for vents is somewhat controversial. Some building scientists believe they are important for air circulation in the cavity and drying. Others argue that the extra water they let in is not compensated for by the improved drying. The following vent details can contribute to an increase in water penetration:

- Vents near windward building corners will admit more water because water is carried with the increased air flow. See Fig. 3 - 5.
- Vents without louvered screens will admit more water because there is no barrier to wind driven rain and water transported by air flow.
- Vents immediately below weepers above may re-admit water that is draining from those weepers.
- Poorly located vents with respect to water flow over special features on the face of the building.

## 3.6  Brick Ties

### 3.6.1  Brick Ties – Strength Behaviour

See Item 3.1.1 – MV/SS Wall as a System – Strength Behaviour for a discussion of tie loads. Refer also to A370-94 Connectors for Masonry[34] for required tie spacings as a function of tie type and Drysdale and Wilson[35] for the results of an extensive testing program on typical MV/SS brick ties.*

### 3.6.2  Brick Ties – Strength Deficiencies/Consequences

See Figure 3 - 14 for an example of a tie that may not have adequate strength.

- The tie is shown mounted on the face of the exterior sheathing at a stud location. Sheathings such as drywall will break down in the presence of moisture and/or load cycling and the ability of the tie to transfer compressive loads to the stud will be compromised. See Drysdale and Wilson[35] for more information. They studied these effects with exterior sheathings of drywall and 38 mm of polystyrene.
- The tie is held in place with a sheet metal screw in pull-out which is a relatively weak connection . A sheet metal screw in pullout has approximately half the strength of the same screw in shear.
- The sheet metal screw is located at a possible condensation site and corrosion may result. The corrosion would severely compromise the strength performance of the screw.

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\*  *The tie stiffness values from the Drysdale and Wilson report have been included in a data base that is part of the MVSS Finite Element computer program. Refer to the User Reference Manual which is a separate publication.*

- The strength (and stiffness) of the tie is severely compromised when the wire pintle is at the outer limits of its adjustment range.
- The distance between the sheet metal screw and the horizontal leg of the connector may be excessive and reduce the strength (and stiffness) of the tie when acting in tension. Note that corrugated strip ties are particularly sensitive to this eccentricity and A370-94[34] restricts this dimension to 6 mm maximum.

Some tie types (corrugated strip ties for example) have little or no adjustment capability and are frequently installed with large bends as a vertical adjustment technique. These large bends will substantially reduce capacity.
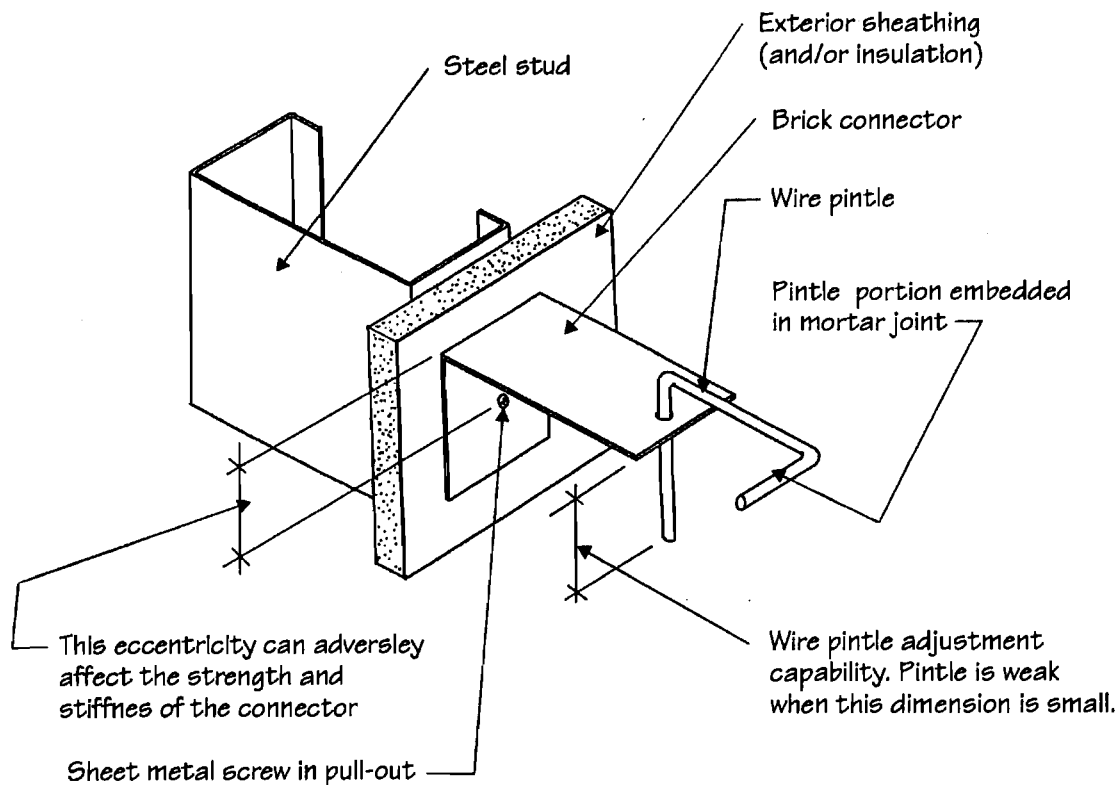
FIGURE 3 - 14       EXAMPLE OF A TIE WITH A NUMBER OF DESIGN FAULTS

In addition, tie strength depends on adequate mortar embedment. A371-94[27] outlines embedment requirements for various tie types. Drysdale and Suter[1] note that

the practice of placing a tie on top of a brick unit without a mortar bed underneath may compromise the tie capacity.

A tied MV/SS wall is a highly redundant system such that the failure or absence of a single tie is not particularly significant. However, if a enough ties fail or are absent, the consequences can range from excessive movement of the veneer (see next Item –Serviceability Behaviour) to the loss of the veneer from the side of the building.

See Item 3.8 – Air Space for a discussion of the effects of large and small air spaces on tie capacity.

See Item 3.21 – Corrosion. Note that ties are in a particularly severe environment with respect to corrosion – subject to frequent wettings and salts. Keller[15] found that the during some months the cavity was almost continuously wet. The practice of using chlorides as an accelerator in the mortar is historically quite common and can significantly increase the rate of corrosion of the embedded portion of the tie. Wind borne chlorides may also be present because of maritime exposure or in urban areas where de-icing salts are used in the winter.

### 3.6.3  Brick Ties – Serviceability Deficiencies/Consequences

Flexible ties will result in larger flexural cracks in the brick veneer and increase the potential for water penetration. See Item 3.1.3, MV/SS Walls as a System – Serviceability Behaviour.

### 3.6.4  Brick Ties – Building Science Deficiencies/Consequences

Mortar droppings on brick ties can form a moisture bridge between the brick veneer and the steel stud backup. The potential for moisture damage to the backup wall is increased if the tie penetrations of the exterior water barrier on the backup wall are not adequately sealed. (Mortar droppings can also increase the time of wetness for the tie and increase the rate of corrosion.)

The thermal bridging effect of brick ties is not significant. Drysdale and Kluge[9] found that during winter conditions stud temperatures were only reduced approximately 5% with the presence of the wrap around style brick tie.

## 3.7  Flashings

### 3.7.1  Building Science Behaviour

Flashings are required to redirect water that enters the cavity space out through

the weep holes in the brick. They are required at any interruption in the wall such as window and door heads, louvers, sills and shelf angles supports.

### 3.7.2  Building Science Deficiencies/Consequences

Inadequate flashings may result in premature shelf angle rusting, moisture migration into the back-up wall and/or moisture migration and consequent efflorescence or freeze-thaw problems into the brick below.

Common flashing deficiencies include:

- Missing at shelf angles, foundation walls, window and door heads, sills and jambs, other wall openings
- Flashing material damaged by construction abuse, by wind prior to brick installation, by shelf angle anchor bolts, by window frame fastenings.
- Inadequate laps and/or seals at laps
- No end dams (End dams are required at discontinuities in flashings such as at the end of a window head detail to insure that any water flowing along the length of the flashing gets directed outside rather than into the wall below.
- Lack of a positive slope to the exterior
- Some flashing materials are inherently poor. They are easily damaged by wind or by installation of brick veneer and may also be unsuitable to form a drip beyond the face of the brick because of susceptibility to UV exposure and/or because they are not stiff enough to maintain a formed shape. A list of suspect flashing materials would include: asphalt impregnated felt, 0.15 mm polyethylene sheet, 30 gauge copper foil laminated to felt or Kraft paper and unreinforced PVC less than 0.70 mm[36, 1].
- Thinner PVC flashings are susceptible to embrittlement due to plasticizer migration as early as two years from time of installation particularly where stressed[36].
- Flashings terminate behind the face of the brick veneer – see Figure 3 – 15 – allowing water to fill the frogs of the brick below (possible freeze-thaw problems)
- Water may run back under the flashing to collect on and corrode the shelf angles. Effective backer rod and sealant are required for effective barrier to water re-entering.
- Flashings not installed behind the exterior sheathing allowing water into the backup wall or onto the shelf angle. See Figure 3 – 15.

See also Drysdale[37], for common flashing faults found in a in a survey of multi-family residential buildings.

Flashing installed behind exterior sheathing.

Where steel stud backup is interrupted by concrete columns or shearwalls, continue flashing and tuck into reglet.

Continue flashing beyond face of veneer
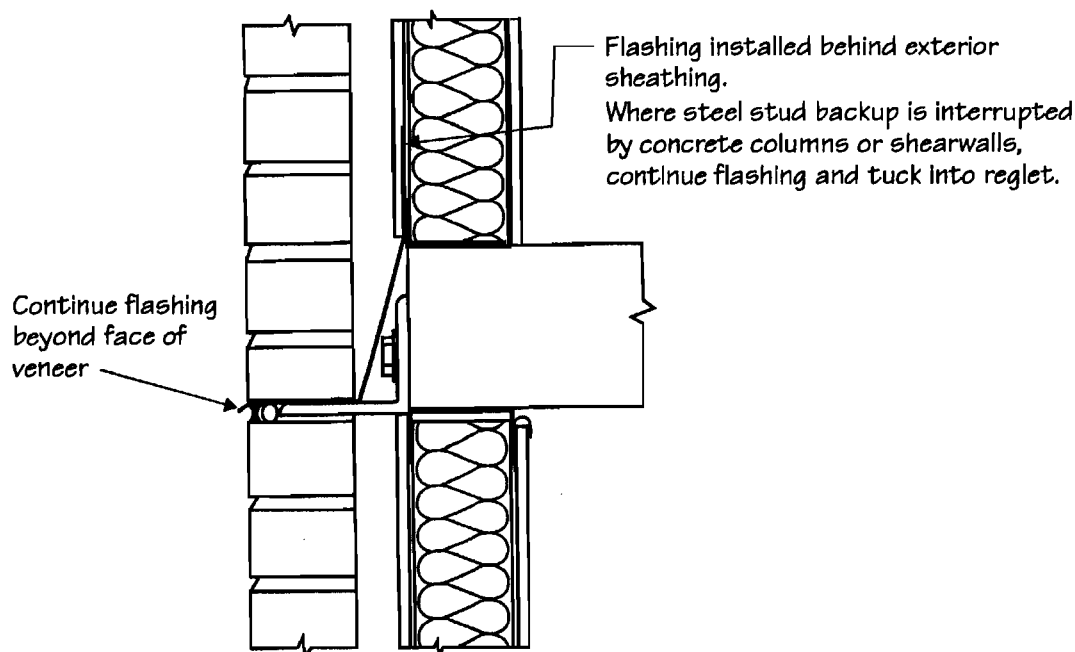
FIGURE 3 - 15        CORRECT FLASHING DETAIL
                     AT SHELF ANGLE

## 3.8  Air Space

### 3.8.1  Air Space – Strength Deficiencies/Consequences

Historically, walls have been designed with a 25 mm nominal air space. (A 50 mm air space represents best current practice.)

In completed buildings, larger and smaller air spaces occur at regular intervals to accommodate dimensional tolerance problems with the structure. Keller[38] in a field investigation of 8 structures across Canada found air spaces ranging from 6 – 64 mm.

Excessively large air spaces can have consequences structurally in that the strength of the brick ties may be reduced. Ties may have reduced buckling capacity because of extra length or less withdrawal capacity from mortar joint because

of inadequate embedment depth.

Note that small air spaces in combination with mortar bridges that bear against the backup can be a structural benefit where the compressive capacity of the brick ties is inadequate. The mortar bridges can act as an alternative load path.

### 3.8.2 Air Space – Building Science Deficiencies/Consequences

See previous Item 3.8.1 for discussion of variations in air space size.

Small air spaces may result in water crossing the cavity to the backup wall. Mortar joint bridges, mortar droppings on ties and accumulated mortar droppings at the bottom of the cavity are all possible paths.*

Where wind pressures fluctuate, large air spaces can increase the air flow requirements through the brick veneer vents and weepers in order to achieve pressure equalization. This effect is, however, relatively minor compared with the effects of leaky air barriers or the absence of vertical baffles to inhibit horizontal air flow in the cavity.

## 3.9 Exterior Insulation and/or Exterior Sheathing

### 3.9.1 Exterior Insulation and/or Exterior Sheathing – Strength Deficiencies/Consequences

Rigid exterior insulation or exterior sheathings such as drywall may be either a deliberate or an accidental air barrier. In either case they are subject to significant wind pressures. Refer to Morrison and Hershfield[39] and Item 3.10. The consequences of inadequate strength include failure of the sheathing/insulation in flexure or more commonly the pull through of fasteners.

For a discussion of exterior drywall as a structural brace for studs see Items 3.13, 3.14.

### 3.9.2 Exterior Insulation and/or Exterior Sheathing – Serviceability Deficiencies/Consequences

Historically, some stud manufacturers have claimed that drywall exterior sheathing in combination with drywall interior sheathing acts compositely with the steel studs to enhance the stiffness of the system and thus reduce overall deflections. This

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\* *Two of the buildings that were investigated by Keller[38] were also reviewed by the writer. Approximately 50% of the running length of mortar joints crossed the air space to contact the sheathing on the steel stud backup.*

design approach is not valid because the composite effect is substantially lost with load cycling and wetting[1, 40].

### 3.9.3 Exterior Insulation and/or Exterior Sheathing – Building Science Behaviour

Exterior insulation is required to keep steel parts above the dew point and to minimize condensation on the inside surface of the exterior sheathing. See Item 3.1.5.2, The Interaction Between Air/Vapour Flow and Thermal Performance.

The exterior sheathing is intended also as a barrier of last resort to water that crosses the cavity.

### 3.9.4 Exterior Insulation and/or Exterior Sheathing – Building Science Deficiencies/Consequences

Laps in the exterior sheathing and tie penetrations are two common locations of water penetration to the backup wall. Some building scientists have argued that for walls with an adequate cavity and minimal mortar bridging, water is unlikely to cross the cavity and the exterior sheathing is better to have a number of penetrations in order to allow the backup wall to dry out in the event of condensation in the stud space. Failure to lap the water resistant barrier over the flashing can result in water penetration to the back–up wall. See Figure 3 – 15.

Absent or inadequate exterior insulation can have the following consequences:
- Condensation on and corrosion of the steel parts (See Corrosion Item 3.21)
- Condensation on the inside surface of the exterior sheathing. Some sheathings such as drywall have limited resistance to repeated wetting and may break down.
- Condensation in the batt insulation and associated problems with mildew and mould growth.
- Condensation on the exterior sheathing and batt insulation may run down to the bottom track resulting in bottom track corrosion.
- Dust shadowing (or in extreme cases condensation) on the interior drywall. The NAHB[41] reported that temperature differences on the interior drywall as little as 2.2 – 2.8°C were sufficient for dust marking at stud locations.

Common deficiencies with exterior insulation include:
- Absent or inadequate thickness
- Failure to extend past the floor to minimize thermal bridging at slab/shelf angle
- Not built tight to the back–up therefore a potential thermal short circuit because of cold air circulation on the warm side of the insulation.

Note that rigid exterior insulations and exterior sheathings may form an accidental air/vapour barrier and inhibit drying out of the backup wall.

## 3.10 Air and Vapour Barriers

### 3.10.1 Overview

The behaviour of air and vapour barriers has been thoroughly documented in the literature[8, 9, 12, 13, 39, 42 - 52] and only a brief review is provided here.

An air barrier must fulfil four requirements:[42]
- low air flow properties
- strong enough to resist the effects of wind and other internal pressures
- durable enough to last the life of the building
- continuous around the building

Theoretically, the air barrier can be located on either the cold or the warm side of the back-up wall. On buildings where the design of an air barrier has been considered, the interior drywall is the most common wall element used for this purpose. Occasionally the exterior sheathing/insulation on its own or the exterior sheathing with an air barrier membrane is used. See Drysdale[37] for the types of air barriers commonly specified as reported in a survey of multi-family residential buildings.

The pressure differences driving air leakage (exfiltration or infiltration) are due to wind, stack effect and pressure equalization. In a cold climate, exfiltration is the more severe problem from a building science perspective and occurs when there is no wind or when the wind adds to the always present exfiltration pressures from stack effect and mechanical pressurization.

### 3.10.2 Air and Vapour Barriers – Strength Behaviour

See Item 3.10.1, Overview.

Air barriers are typically designed to resist the full wind load on the wall of the building. Some[39] have suggested that stack effect pressures and the pressures due to mechanical pressurization should be superimposed on these pressures.

Vapour barriers are subject to very small pressures differences (assuming they are not also accidental air barriers) that are of little consequence structurally.

### 3.10.3 Air and Vapour Barriers – Strength Deficiencies/Consequences

Where the interior drywall is used as an air barrier it must have sufficient strength to span from stud to stud while carrying the design wind loads. Studies and tests show failures at fastener locations with the heads of the drywall screws pulling through or flexural failures of the board itself.

Flexural failures are characterized by a delamination and tearing of the paper and a crack forming through the thickness of the drywall. These flexural failures may occur at stud locations or between studs. Note that drywall has a higher flexural strength in the long direction than in the short direction. Therefore, board installed with its long dimension in the horizontal direction will be significantly stronger where flexural strength is the critical failure mechanism.

The exterior sheathing/insulation is subject to similar failure mechanisms as the interior drywall. Refer to Morrison and Hershfield[39].

Air barrier membranes can be applied to the exterior sheathing (trowelled on, torched on or stuck on) and are subject to debonding type failures under the action of wind.

On some buildings, either by accident or design, the primary air barrier is the polyethylene vapour barrier material which, unless supported on either side, has inadequate strength to perform as an air barrier. Failures typically occur at laps and connections to other air barrier elements.

Note that air barriers with low leakage rates are subject to higher loads than air barriers with high leakage rates.

### 3.10.4 Air and Vapour Barriers – Serviceability Deficiencies/Consequences

Some air barriers (such as polyethylene) in addition to strength problems also deform considerably when subject to wind pressures. This deformation adversely affects the pressure equalization for the wall.

Air barriers are also subjected to continuous albeit lower pressures due to stack affect and mechanical pressurization. These continuous pressures can adversely affect air barrier materials susceptible to creep deformations.

Air barriers are also subject to large local deformations at movement joints. For example, the steel stud system is typically installed with a movement joint at the top to accommodate slab defections without loading the studs axially. The interior drywall air barrier requires special detailing consideration to accommodate this movement. See Figure 3-19, inner and outer top track detail.

### 3.10.5 Air and Vapour Barriers – Building Science Behaviour

See Item 3.10.1, Overview.

As discussed in Drysdale and Suter[1], much of the historic difficulty with air and vapour barriers is related to the misunderstanding of the independent functions of air and vapour barriers. Vapour barriers are intended to control the flow of water

vapour through materials whereas air barriers are intended to limit the flow of air through the wall. See also Quirouette[43].

Drysdale and Suter[1] argue that it is very difficult to insure that either barrier performs only a single function and therefore, a common problem may be that many wall assemblies contain essentially a double set of barriers. In practice, this means that moisture may become trapped between the barriers.

### 3.10.6 Air and Vapour Barriers – Building Science Deficiencies/Consequences

Leaky air barriers can result in large volumes of air and moisture entering the back-up wall[9]. Condensation and moisture problems can result particularly when temperatures inside the stud space fall below the dew point. See Item 3.1.5.2, The Interaction Between Air/Vapour Flow and Thermal Performance. Leaky air barriers can also result in high humidities and condensation in the cavity.

Leaks typically occur at:
• Inadequate joints in the air barrier material
• Inadequate Interfaces with other air barrier elements (such as windows or concrete slabs and intersecting shearwalls)
• Discontinuities in the air barrier in concealed spaces (such as the interruption of the interior drywall wall air barrier by intersecting drywall partitions, behind radiators, behind cupboards, above suspended ceilings)
• Penetrations through the air barrier (such as brick ties through exterior air barriers or electrical boxes through interior air barriers).

Condensation due to leaky air barriers may result in:
• Corrosion of the steel parts – steel framing members and fasteners in the stud space and brick ties in the cavity (See Item 3.21, Corrosion)
• Wetting and deterioration of the interior and/or exterior sheathing
• Wetting of the batt insulation and associated mould and mildew growth
• Moisture from the exterior sheathing and batt insulation running down to the bottom track resulting in bottom track corrosion (See Item 3.21, Corrosion)
• Condensation on the backside of the brick resulting in freeze-thaw spalling. See Item 3.5.4.2.
• Efflorescence on the outside face of the brick. See Item 3.5.4.1.

Other symptoms of a leaky air barrier include:
• Dust particle accumulation in the batt insulation (filtered from the exfiltrating air)
• Cold drafts during periods of infiltration of exterior air (positive wind pressure)

Note that the presence of an accidental air/vapour barrier on the exterior side of the stud space can have a serious negative impact on the ability of the backup wall to dry out. The following is a quote from Drysdale and Suter[1]:

"Tests, analysis and field review have all demonstrated that the mass of water

which can be carried into a wall by air leakage even through a very small hole is several magnitudes larger than the amount of water which is transmitted through a wall by diffusion. Therefore, when the airborne moisture condenses, it is quite unlikely that this can be removed in sufficient volume by vapour transmission to the outside. In fact, drying out is more likely to be accomplished by dry air moving through the wall under different weather conditions."

The philosophy of allowing exterior barriers provided they have adequate vapour permeability should be approached with caution.

Note that relatively large holes can be tolerated in the vapour barrier without consequence because the quantity of moisture moved by vapour diffusion is small compared with the quantity of moisture moved by air flow through the air barrier.

## 3.11 Stud Space Insulation

### 3.11.1 Stud Space Insulation – Building Science Behaviour

Stud space insulation is used to enhance the R–value of the wall without increasing the overall wall thickness.

### 3.11.2 Stud Space Insulation – Building Science Deficiencies/Consequences

The presence of stud space insulation has the effect of adversely altering the thermal profile through the wall such that the dew point temperature may fall within the stud space. See Item 3.1.5.2, The Interaction Between Air/Vapour Flow and Thermal Performance. Exterior insulation is required to shift the dew point towards the outside.

The thermal performance of the stud space insulation can be compromised by gaps. Gaps typically occur because the batts sag, are not thick enough to fill the entire stud space or are not wide enough to fill inside the stud profile. Note that batts for wood stud construction are dimensioned for the clear inside distance between studs whereas batts properly dimensioned for steel stud construction are wider – typically equal to the centre to centre stud spacing.

## 3.12 Interior Sheathing

### 3.12.1 Interior Sheathing – Strength Deficiencies/Consequences

Rigid interior sheathings such as drywall may be either a deliberate or an accidental air barrier. In either case they are subject to significant wind pressures. Refer to Morrison and Hershfield[39] and Item 3.10. The consequences of inadequate strength include failure of the sheathing/insulation in flexure or more

commonly the pull through of fasteners.

For a discussion of interior drywall as a structural brace for studs see Items 3.13 and 3.14.


### 3.12.2 Interior Sheathing – Serviceability Deficiencies/Consequences

See 3.9.2.


### 3.12.3 Interior Sheathing – Building Science Behaviour

The interior sheathing is commonly used as the primary air barrier element. See Item 3.10, Air Barriers.


### 3.12.4 Interior Sheathing – Building Science Deficiencies/Consequences

See Item 3.10, Air and Vapour Barriers


## 3.13 Typical Stud


### 3.13.1 Typical Stud – Strength Behaviour

Studs are simply supported flexural members that span vertically from slab to slab and are subjected to wind loads transferred to the studs via brick ties or attached air barriers. These wind loads are typically eccentric with respect to the shear centre of the studs and torsion therefore results. Secondary bracing is used to resist the torsional component of the load and the tendency for the studs to buckle laterally.

See CSSBI[2] for further discussion of stud structural behaviour and Appendix B for configuration and cross sectional geometry. See also Item 3.1, MV/SS Walls as a System and Item 3.14, Bridging.


### 3.13.2 Typical Stud – Strength Deficiencies/Consequences

Stud strength is a function of the thickness of the material, the geometry of the stud profile (depth, flange, lip, inside bend radius and cut-out size and spacing), the yield strength of the material and secondary bracing.

Inadequate flexural strength will result in a midspan failure typically characterized by twisting and a local buckle in the vicinity of a web cut-out. This is a potentially dangerous failure that may lead to the collapse of the entire wall.

Inadequate web crippling strength will result in a local failure in the bend radius at the web to flange intersection. The failure may be at the ends of the stud due to the end reaction from the top or bottom track or along the length of the stud due to a high local load from some brick tie types. The end web crippling strength is considerably reduced if a web cut-out is too close – 300 mm from the centreline of the last cut-out to the end of the stud is the industry standard minimum. End web crippling failures are unlikely to result in the collapse of the overall wall whereas web crippling failures along the length of the stud interact unfavourably with moment which may lead to a premature flexural failure.

Without adequate torsional restraint from secondary bracing, studs will fail prematurely in a torsional-flexural mode. For further discussion on torsional restraint, see CSSBI[2] and Item 3.15, Bottom Track, Item 3.16, Top Track Assembly. and Item 3.14, Bridging.

The flexural strength of studs is sensitive to local damage. For example, local denting of the product (say in the order of 2 – 4 times the thickness of the material) or damage to the lip stiffener may significantly reduce its flexural capacity.

Inexperienced welders can compromise the capacity of the stud member by burning through at the location of the weld. Burn through is most serious in the region of the flange at or near midspan.

The weakening effect of pre-punched cut-outs should be accounted for in the manufacturers load tables but frequently is not. Compared with unperforated product, industry standard cut-outs typically result in moment reductions in the order of 10% and widely varying reductions in shear – 50% is not uncommon. Shear may be a concern with highly loaded built-up window jamb members. or cantilevering parapet details where combined moment and shear may be an issue. See Item 3.19, Parapets.

See Item 3.21, Corrosion

### 3.13.3 Typical Stud – Serviceability Behaviour

See Item 3.1, MV/SS Wall as a System.

### 3.13.4 Typical Stud – Serviceability Deficiencies/Consequences

See Item 3.1, MV/SS Wall as a System.

### 3.13.5 Typical Stud – Building Science Behaviour

See Item 3.1.5.2, The Interaction Between Air/Vapour Flow and Thermal Performance for a discussion on thermal bridging.

## 3.14 Bridging

### 3.14.1 Bridging – Strength Behaviour

Three types of bracing are commonly used to resist the torsional component of the load and the tendency of the studs to buckle laterally. These are:

- Interior and exterior sheathings acting as diaphragms that transfer the bracing forces to the top and bottom tracks.
- Steel strap face bridging attached to the inner and outer stud flange (welds or screws).  See Figure 3 – 16. The accumulative bracing forces are transferred by periodically connecting the flat strap bridging to the main structure or by providing blocking–in every few stud spaces.
- Through–the–knockout bridging with welds or screws. See Figure 3 – 17 for a typical screwed detail. Twisting of an individual stud is resisted by the bridging channel which mobilizes the major axis bending strength of the adjacent studs.

The maximum spacing for steel bridging is 1500 mm o.c. based on standard industry practice or 1200 mm o.c. based on the recommendations in Drysdale and Breton[4] and Drysdale and Suter[1].

See CSSBI, Lightweight Steel Framing Design Manual[2] for more detail.

### 3.14.2 Bridging – Strength Deficiencies/Consequences

#### 3.14.2.1 *Sheathing*

The following deficiencies will compromise the effectiveness of sheathing as a brace:

- Sheathing on one side is not effective unless supplemented by steel bridging on the other side. Most rigid insulation materials do not have sufficient diaphragm strength to act as a brace.
- Gypsum wallboard sheathings will restrain studs in thinner material (0.91 mm – 20 gauge)[4] but require supplementary steel bridging to effectively restrain studs in thicker material.
- When subjected to wetting, the bracing performance of gypsum wall board deteriorates significantly[4]. Note that exterior drywall sheathings may be in a wet environment due to condensation on the inside surface

and wetting on the outside surface from rain entering and bridging the cavity.

Gypsum wall board sheathings are generally regarded as unreliable. It is industry practice to supplement gypsum drywall sheathings with steel bridging to insure that the wall has the necessary structural integrity.

### 3.14.2.2 *Steel Strap Face Bridging*

Steel strap face bridging is designed to act only in tension. The most common deficiency is inadequate or absent anchorage or blocking-in. Note that considerable bridging forces can accumulate in strap bridging.

Moderate slackness in strap bridging is normal and does not compromise its performance.

See Figure 3 – 16

Steel stud

Steel strap
face bridging

Sheet metal screw
(or weld) strap to stud
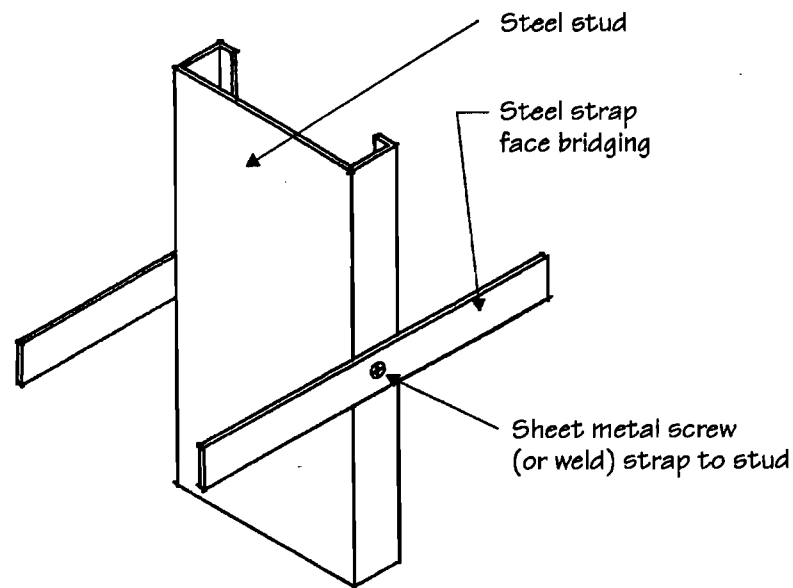
Note: Periodic anchorage of straps and/or blocking-in
every few stud spaces is required.

FIGURE 3 - 16        STEEL STRAP FACE BRIDGING

### 3.14.2.3 *Through-the-knockout Bridging*

Through-the knockout bridging requires careful detailing to perform.
There are a number of recurring deficiencies.

- Interior drywall installers, unfamiliar with structural stud, may install the bridging channel through the stud knock-outs with no mechanical connection – relying instead on friction fit. As a torsional restraint, this type of detail is structurally useless.
- A similar error is made when only one screw (instead of two) is installed to connect the bridging channel to the bridging clip angle. The connection will behave as a hinge, unable to resist the twisting in the stud.
- Sometimes the bridging is discontinued where the stud knockouts do not align or the bridging may be inadequately spliced.

- The thickness of the bridging clip angle may be inadequate. As a minimum 1.22 mm (18 gauge) with 1.52 mm (16 gauge) preferred is required to provide adequate stiffness at the connection.

Note that anchorage of through-the-knockout bridging is not as critical as for steel strap face bridging.

See Figure 3 - 17
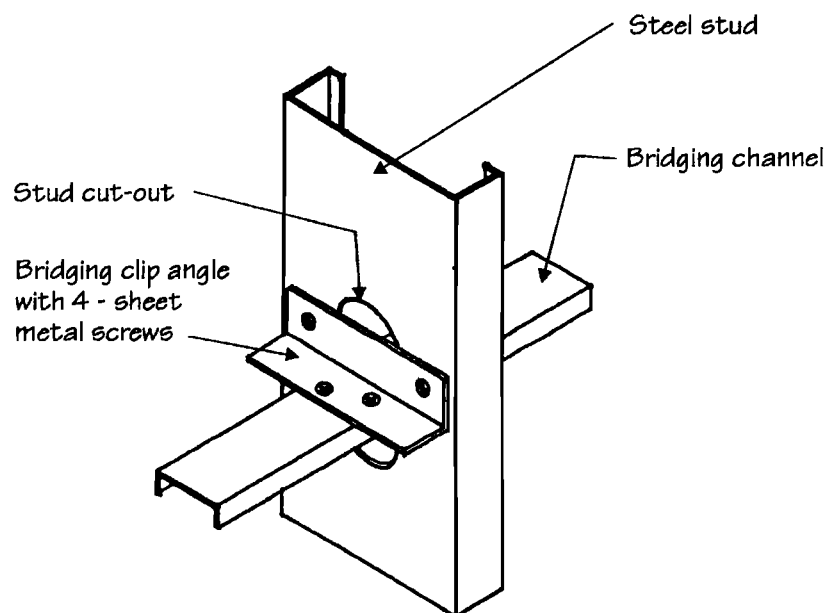


FIGURE 3 - 17          THROUGH-THE-KNOCKOUT
                                BRIDGING

## 3.15 Bottom Track

### 3.15.1 Bottom Track – Strength Behaviour

See Figure 3 - 18, B-2 and B-3 (Appendix B).

The bottom track is designed as a continuous flexural member loaded by the stud end reactions and spanning from concrete fastener to concrete fastener.

The stud end reaction is resisted in bearing by the upstanding leg of the track which must have adequate resistance to develop the web crippling strength of the stud. Drysdale and Breton[4] found that if the thickness of the bottom track is equal to or greater than the thickness of the stud, then the web crippling strength of the stud can be developed. More specific design guidelines have been developed since[53].

Drysdale and Breton[4] found that concrete fastener spacings not more than 800 mm o.c. were required to control track deformations and local buckling.

Connections (sheet metal screws or welding) are required between the track and both the inner and outer stud flanges.

See CSSBI[2] for more detail on the structural behaviour of this connection.
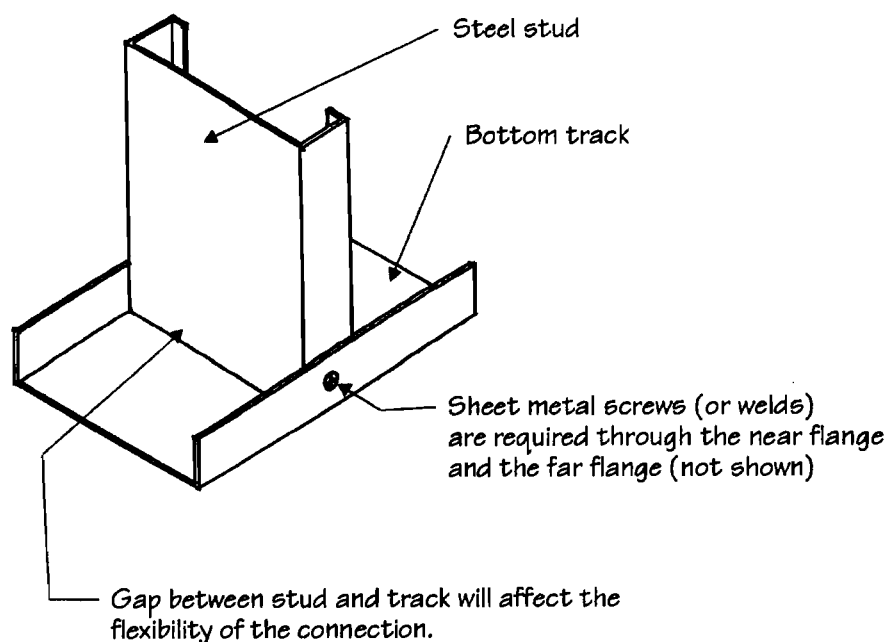
FIGURE 3 - 18       STUD TO BOTTOM TRACK CONNECTION DETAIL

### 3.15.2 Bottom Track – Strength Deficiencies/Consequences

Common Deficiencies include:

* Absent, infrequent or inappropriate concrete fasteners. Concrete fastener spacings should not exceed 800 mm o.c.[1] and additional fasteners may be required where loads are high such as at window jambs. Some types of fasteners are in common use but may not be appropriate for this type of construction – powder actuated pins, for example. The capacity of all concrete fasteners is sensitive to edge distance – see CSSBI[2] or the fastener product literature for more detail. Note that identifying concrete fasteners already in place can sometimes be difficult and removal of a fastener and the surrounding concrete may be necessary for this purpose.

Bottom track concrete anchors are usually assumed to be loaded only in shear (no pull-out due to prying forces).

Poor concrete anchoring is a potentially dangerous situation that could lead to the collapse of the entire wall.

- Inadequate track thickness will result in excessive local deformations in the track but rarely overall failure provided concrete fastening is adequate.

- Absent sheet metal screws (or welds) between the track and the studs. Both the inner and outer stud flange should be connected to the track flanges otherwise the necessary torsional restraint for the stud (and also the track) will not be developed.

- Bottom tracks typically experience a greater time–of–wetness than other elements of the back–up wall (with the exception of brick ties) and are therefore more inclined to corrode – track, sheet metal screws and concrete connectors are all susceptible. The wetness is typically due to condensation in the backup wall or rain penetration from the outside. See Item 3.21, Corrosion.

### 3.15.3 Bottom Track – Serviceability Deficiencies/Consequences

The gap between the end of the stud and the bottom track does not affect the ultimate strength of the connection but can dramatically increase the flexibility of the connection. Drysdale and Breton[4] studied a number of combinations of end gap and stud and track material thickness. Refer to the report for more detail.*
The industry standard for new construction is to limit the end gap to 4 mm. See CSSBI S5[54].

Flexible bottom tracks have little impact on brick behaviour. See Drysdale and Breton[4] for more information.

### 3.15.4 Bottom Track – Building Science Behaviour

See Item 3.1.5.2, The Interaction Between Air/Vapour Flow and Thermal Performance for a discussion on thermal bridging.

## 3.16 Top Track Assembly

### 3.16.1 Top Track Assembly – Strength Behaviour

The top track detail is designed to resist the end reactions from the wall studs and at the same time provide a movement joint such that slab deflections do not load

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*    *The MVSS finite element computer program that is part of this project includes a data base with the experimentally derived stud to track flexibilities from the Drysdale and Breton work.*

the stud axially.

There are several different details in use that satisfy the structural requirements for top track assemblies. The most common is the inner and outer top track detail which will be discussed here. See Figure 3 – 19. Drysdale and Breton[4] tested this detail and a number of others including slotted clip angles, flexible clips, boxed track and the single outer track. Refer to Drysdale and Breton[4] and CSSBI[2] for more information on the structural behaviour of this connection.

Structurally, the inner top track behaves as a torsional restraint for the studs and as a load spreading device to minimize concentration of load on the outer top track. The outer top track leg behaves as a cantilever to resist the lateral forces imposed by the inner top track.

### 3.16.2 Top Track Assembly – Strength Deficiencies/Consequences

Common deficiencies include:

- Absent, infrequent or inappropriate concrete fasteners. Refer to Item 3.15, Bottom Track, for more discussion on these fasteners. Note that the loading on top track fasteners is more severe than for bottom track fasteners because pull-out due to prying interacts unfavourably with shear. The cut-outs shown in Figure 3 – 19 in the inner top track are usually not present and inspection of the existing fasteners is difficult. Usually, it is necessary to cut inspection holes in the inner top track.
- Inadequate thickness of outer top track can compromise the structural integrity of this connection. The leg of the top track, loaded as a cantilever from the underside of the slab above, must have adequate thickness (and yield) to resist the lateral forces imposed.
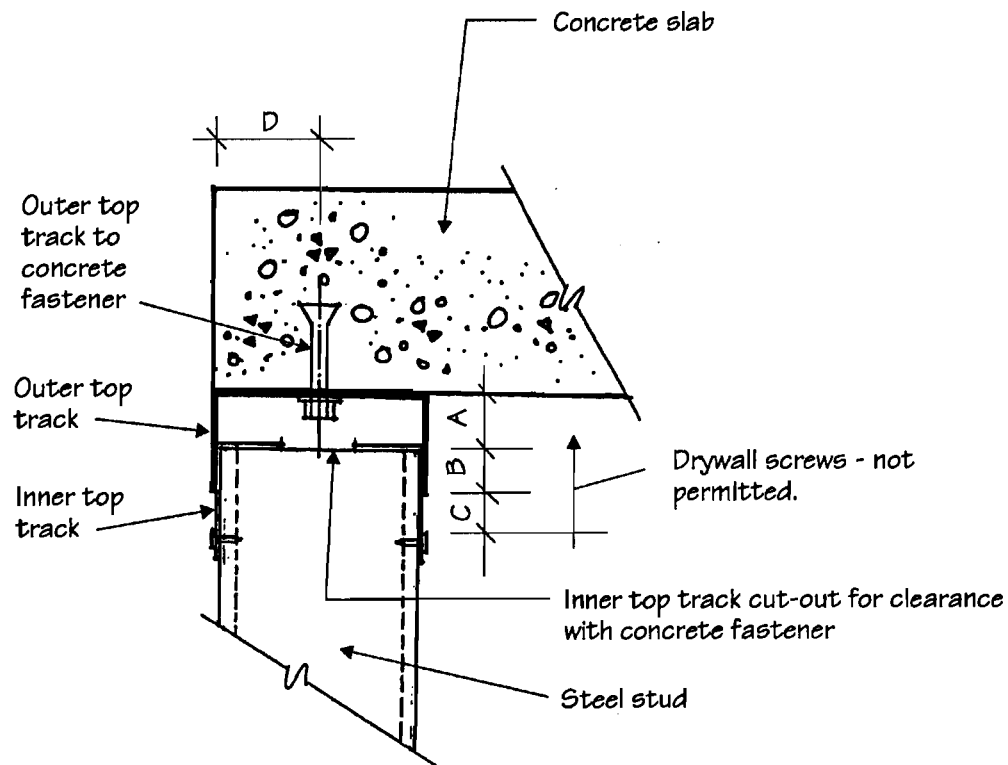
**Notes:**

1. Dimension "A" accomodates slab deflections above to avoid
   loading the stud axially.
2. Dimension "B" is the engagement between the inner and the outer track.
   If this dimension is small, any deflection of the slab below or rebound
   of the slab above may result in the complete disengagement of the backup wall.
3. Dimension "C" should be greater than or equal to "A" otherwise the movement
   will be compromised.
4. The strength of concrete fasteners is a function of fastener type, fastener
   diameter, embedment depth, concrete strength, centre to centre spacing
   and dimension "D", the edge distance.

FIGURE 3 - 19      INNER AND OUTER TOP TRACK
                               MOVEMENT JOINT DETAIL

- Inadequate overlap between the inner and outer top track (Dimension "B" Figure 3 – 19). This detail can be used to a certain extent to accommodate dimensional variations due to construction as well as behave as a movement joint. Occasionally, where tolerances are excessive there may be very little overlap between the inner and outer tracks. This is potentially a very dangerous situation since only a small amount of deflection in the slab below may mean the complete disengagement of the tracks and the almost certain loss of the entire wall.

- Inadequate gap between the inner and outer top track (Dimension "A" Figure 3 – 19) such that slab deflections may translate into axial loads on the studs for which they are not designed.

- The movement joint capability of the inner and outer top track detail can be defeated by screwing the connection shut at the time the drywall is installed. Drywall screws should be installed no higher than the sheet metal screws shown in Figure 3 – 19 that connects the inner top track to the stud.

- Because the inner top track acts as a torsional restraint for the stud, missing sheet metal screws (or welds) between the inner top track and the stud may result in a premature torsional failure of the stud.

See Item 3.21, Corrosion.


### 3.16.3 Top Track Assembly – Serviceability Deficiencies/Consequences

This is an inherently flexible connection when loaded by wind. Excessive movement may fail the caulking in the soft joint and may overstress the air barrier material that traverses this connection detail.

Flexible top tracks have little impact on brick behaviour. See Drysdale and Breton[4] for more information.


### 3.16.4 Top Track – Building Science Behaviour

See The Interaction Between Air/Vapour Flow and Thermal Performance (Item 3.1.5.2) for a discussion on thermal bridging and Figure 3-7 which illustrates a particularly severe thermal bridge that can form through the outer top track when in contact with the shelf angle.


## 3.17 Built-up Jamb Studs


### 3.17.1 Built-up Jamb Studs – Strength Behaviour

Jamb studs are special vertical stud members that frame either side of window openings and carry the wind load from the window area to the top and bottom tracks. The behaviour of jamb stud is similar to typical studs with some

differences.

Jamb studs carry more load than typical stud members and some form of rein-
forcement is therefore required. Typical built-up member details are shown in
Figure B – 2 and in Figure 3 – 20 for welded and screwed construction. See also
CSSBI[2] for information on design and detailing jamb members.

Steel studs welded toe-to-toe
to form built-up jamb member.

It is difficult to connect studs
toe-to-toe in screwed construction.

Steel stud reinforced with
screwed on track to form
a built-up jamb member.

Reinforcing angle where web
crippling is critical.

Track

3 - 20a
Welded Detail

3 - 20b
Screwed Detail
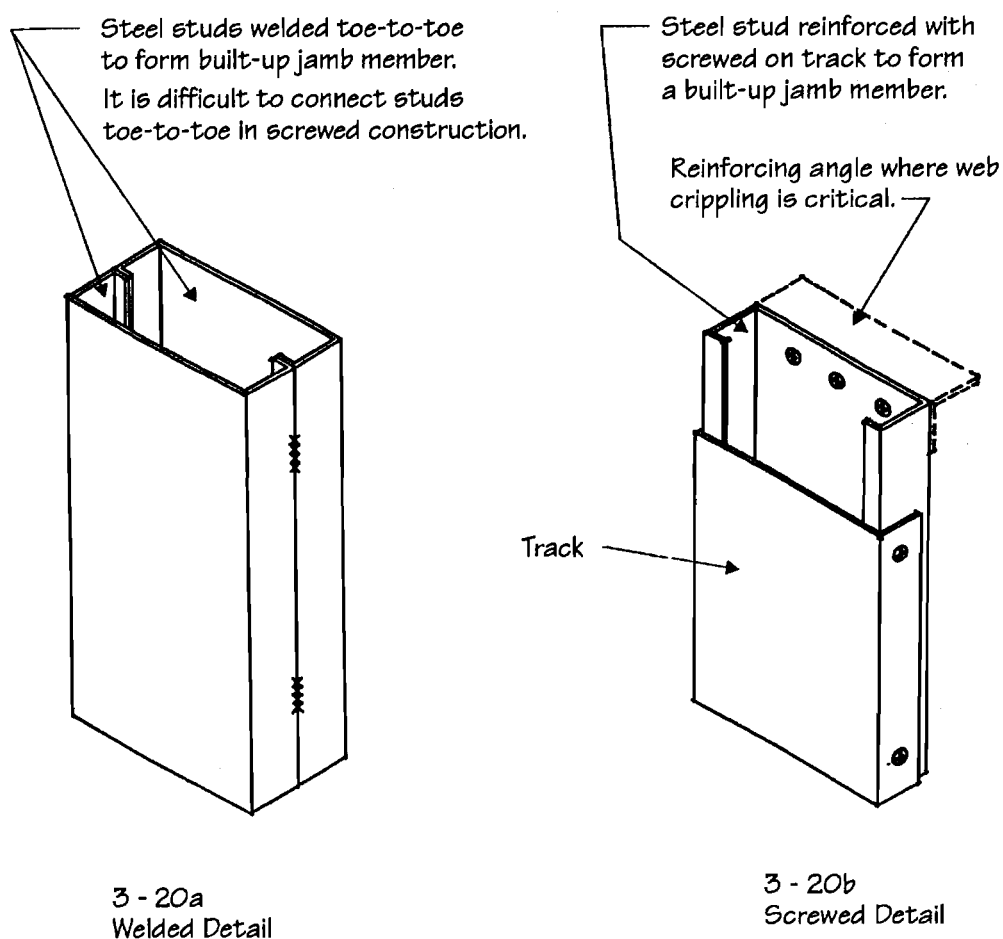
FIGURE 3 - 20          BUILT-UP JAMB DETAILS

### 3.17.2 Built-up Jamb Studs – Strength Deficiencies/Consequences

Common deficiencies include:

- The built–up jamb members may be inadequately interconnected. In screwed construction, two studs toe to toe are not recommended as a built–up member because it is difficult to adequately connect the studs together. See Figure 3 – 20a for toe to toe studs welded. Figure 3 – 20b illustrates the type of detail better suited to screwed construction.
- Refer to Typical Stud – Strength Deficiencies/Consequences, Item 3.13.2.

See Item 3.21, Corrosion, and CSSBI$^2$ for more information about designing and detailing these members.

### 3.17.3 Built–up Jamb Studs – Serviceability Behaviour

See Item 3.1, MV/SS Wall as a System.

### 3.17.4 Built–up Jamb Studs – Serviceability Deficiencies/Consequences

See Item 3.1, MV/SS Wall as a System.

### 3.17.5 Built–up Jamb Studs – Building Science Behaviour

See Item 3.1.5.2, The Interaction Between Air/Vapour Flow and Thermal Performance for a discussion on thermal bridging.

## 3.18 Head and Sill Members

### 3.18.1 Head and Sill Members – Strength Behaviour

Head and sill members are horizontal flexural members that transfer wind loads from the window area to the jamb studs. For typical punched window dimensions, a single track profile is generally adequate. For larger openings, a built–up member may be required.

The sill track typically carries the weight of the window and requires jack studs below for support. See Figure B – 2. See CSSBI$^2$ for more information on designing and detailing these members.

### 3.18.2 Head and Sill Members – Strength Deficiencies/Consequences

Common deficiencies include:
- Inadequate flexural strength. The problems associated with sill and head tracks are similar to those for studs except that there are no cut–outs and the end

connections are not susceptible to web crippling type of failures. See Item 3.13.2, Typical Stud – Strength Deficiencies/Consequences.

- Inadequate or missing end connection. There are a variety of acceptable methods to connect the sill and head members to the jamb members. See CSSBI for connections using clip angles.
- Inadequate interconnection of built-up members.

### 3.18.3 Head and Sill Members – Serviceability Behaviour

See Item 3.1, MV/SS Wall as a System.

### 3.18.4 Head and Sill Members – Serviceability Deficiencies/Consequences

See Item 3.1, MV/SS Wall as a System.

### 3.18.5 Head and Sill Members – Building Science Behaviour

See Item 3.1.5.2, The Interaction Between Air/Vapour Flow and Thermal Performance for a discussion on thermal bridging.

## 3.19 Parapets

### 3.19.1 Parapets – Strength Deficiencies/Consequences

Parapet strength deficiencies are usually associated with the steel stud backup. Note that some parapets are intended to accept substantial vertical and horizontal loads from window washing equipment, swing stages etc.

The detail shown in Figure 3 – 21b is typically fastened to the roof slab with concrete fasteners through the bottom track. This detail provides neither a strong or a stiff connection. (This detail will work if anchor plates are cast into the roof slab at regular intervals and hot–rolled angles, channels or hollow structural sections welded to the anchor plates are cantilevered over the height of the parapet to support the top track.)

The detail shown in Figure 3 – 21a is a common solution to the parapet problem but is dependent on a slip connection between the studs and the roof slab. A number of connection details used in this application have questionable capacity. In addition, combined bending and shear stress at the roof level may be critical particularly if the there is a web cut–out at or near this location. See CSSBI[2] for more information on designing and detailing parapets

### 3.19.2 Parapets – Serviceability Deficiencies/Consequences

Parapets require more frequent vertical movement joints because they are subjected to temperature extremes on both faces. Drysdale and Suter[1] recommend a maximum joint spacing of 6 m as opposed to 12 m for the veneer below. See Figure 3 – 9 and 3 – 10 for typical cracking patterns that result from inadequate vertical movement joints.

Steel stud continuous past the roof slab

Slip connection detail required here

Roof slab
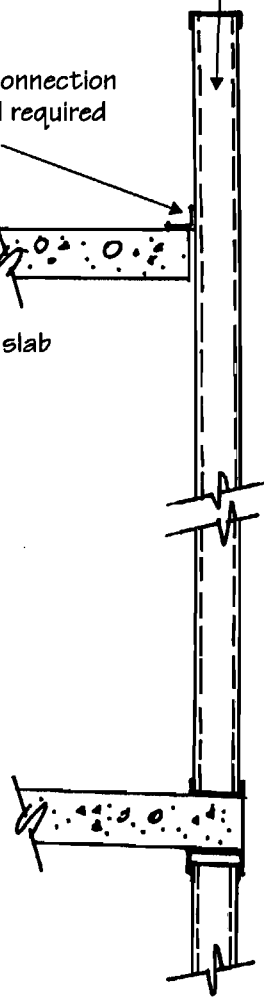
3 - 21a

Steel stud wall cantilevering from the top of the roof slab.

Fastening the bottom track to the concrete roof slab creates neither a strong nor a stiff connection.

Roof slab
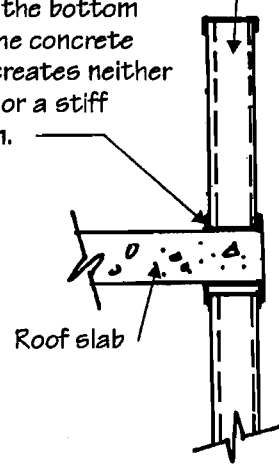
3 - 21b

FIGURE 3 - 21      STEEL STUD PARAPET DETAILS

### 3.19.3 Building Science Deficiencies/Consequences

Parapets are subject to a number of special moisture problems – particularly since they are subject to wind driven rain on three faces:
- Leakage through the cap flashing at flashing joints.
- Leakage under the vertical leg of the cap flashing due to wind driven rain. Drysdale and Suter[1] recommend a vertical leg of 75 mm.
- Leakage at the parapet to roof junction.
- Condensation inside the parapet because of vertical currents of moist air being carried into parapet from the cavity below (or the stud space below for the case of stud details that are continuous past the roof slab).
  These moisture problems typically manifest themselves as efflorescence, spalling, ice lenses at the parapet level or water penetration into the wall below.

## 3.20 Sealants

### 3.20.1 Sealants – Serviceability Deficiencies/Consequences

Sealants are used to prevent air and moisture from penetrating joints in the building materials without inhibiting the differential movement between the components being joined.

Common deficiencies include:
- Inadequate movement capability resulting in rupturing or peeling under repeated cycles of extension or compression. Note that inadequate movement joint width will defeat any caulking material and can result in the caulking material being squeezed out of the joint due to a one time cyclical movement.
- Poor adhesion to the surfaces being joined resulting in de-bonding from one or both surfaces.
- Improper sealant profile or depth. A properly installed sealant bead should have concave surfaces and a depth not more than half the joint thickness.
- Improper backer materials. Backing should not bond with the sealant and should be as compressible as the sealant. Note that some backing materials can transmit sizeable compressive stresses that will defeat the ability of the movement joint to close.
- Premature ageing resulting in hard and inflexible caulking material. This may be the result of poor quality materials or improperly mixed multi-component sealant material at the time of installation.

Refer to Drysdale and Suter[1] for recommendations on good sealant practice.

## 3.21 Corrosion

The mechanisms of corrosion are described in considerable detail in S478[55] which also

includes a methodology for establishing a corrosion severity rating.

Resistance to corrosion has historically been provided in a prescriptive way by requiring minimum standards of coatings (primarily zinc) on steel products. Little or no effort has been made to link the severity of the micro-climate in the wall to protective coating requirements.

### Steel Stud Backup

Steel studs are typically manufactured from coils of sheet product that have been hot dipped galvanized at the mill and subsequently slit to the desired blank widths, roll formed and punched . The cut edges are therefore not galvanized but it is generally accepted that the neighbouring zinc will protect the cut edges sacrificially.

The durability of the stud member in a corrosive environment is primarily a function of the thickness of the zinc coating rather than the thickness of the steel. Current industry standard practice is to supply stud products and accessories in Z275 galvanizing (G90) which means that there are 275 gm/m² of zinc, summed on both sides. Specifications[34] typically call up a lighter Z180 (G60) coating which is the lightest possible coating that leaves some free zinc on the surface. Free zinc can be observed on new product in the form of spangles (zinc crystals).

Lighter coatings such as wiped coat (hot-dipped with the free zinc removed) and electro-galvanized are generally considered inadequate for exterior wall construction. Occasionally in older buildings, painted product can be found.

### Brick Ties

The coating requirements for brick ties have been substantially upgraded in A370-94[34]. For exterior walls above grade, this standard establishes coating requirements as a function of building height and the driving-rain index.

As a minimum in both the current and previous A370[34, 56], steel parts are required to be hot-dipped galvanized after fabrication. (A370-94 requires stainless steel ties where level III protection is required.)

Note that a number of ties in use in older buildings do not meet the hot-dipped requirement. It is not uncommon to find corrugated strip ties with only an electro-galvanized coating. Some two component ties are supplied with the wire pintle hot-dipped but the connector plate that anchors to the stud only in Z180 or Z275 galvanizing. Note that Z275 galvanizing has less than a third of the zinc necessary if a hot-dipped part is required.

### Shelf Angles

While some shelf angles are supplied with hot-dipped galvanizing, most are supplied as a painted product only and are therefore subject to corrosion. Good detailing is required to reduce the time-of-wetness for the shelf angle – primarily good flashings and proper

sealants in horizontal movement joints.

### *Extent of Corrosion*

A corrosion diagnosis should be accompanied by an estimation of its seriousness. Three levels of corrosion are possible:

- White rust is the result of oxidation of free zinc on the surface of the member and usually means some zinc has been lost but some remains.
- Brown rust results when all of the protective zinc has been lost and the parent material is starting to corrode. Brown rust can vary from a few isolate spots to a substantial portion of the area of the part. Brown rust is usually reported in terms of the percentage of affected area.
- Brown rust with loss of parent material is the most severe condition and is usually a concern structurally.

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31   *Efflorescence, Prevention and Control (Technical Note No. 23A Revised June 1985), Brick Institute of America Technical Notes on Brick Construction*

32   *CBD 2 Efflorescence, by T. Ritchie, Canadian Building Digest, Institute for Research in Construction, National Research Council of Canada, 1960*

33   *Frost Damage to Clay Brick in a Loadbearing Masonry Building, by A.H.P. Maurenbrecher and G.T. Suter, Canadian Journal of Civil Engineering, April, 1993*

34   *A370-94 Connectors for Masonry, Canadian Standards Association.*

35   *A Report on Behaviour of Brick Veneer/Steel Stud Tie Systems, by R.G. Drysdale and M.J. Wilson, McMaster University, for Canada Mortgage and Housing Corporation March 1989.*

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37   *Construction Problems in Multi-Family Residential Buildings, by R.G. Drysdale, for Ontario New Home Warranty Plan and Canada Mortgage and Housing Corporation, March 1991.*

38   *Field Investigation of Brick Veneer/Steel Stud Wall Systems, by Suter Keller Inc., for Canada Mortgage and Housing Corporation, Ottawa, November 30, 1989.*

39   *Structural Requirements for Air Barriers, by Morrison Hershfield Limited, for Canada Mortgage and Housing Corporation, August 13, 1991.*

40   *Performance Evaluation of Brick Veneer with Steel Stud Backup, by J.O. Arumala and R.H. Brown, Clemson University, for the Brick Institute of America and the Metal Lath/Steel Framing Association, April 1982.*

41   *Status of Cold-Formed Steel Framing in Residential Construction - Thermal Performance in Exterior Walls, by NAHB Research Foundation Inc., for American Iron and Steel Institute, November 1973*

[42]     *Air Barriers: Assemblies and Construction Materials - Part 1, by J. Rousseau, National Building Envelope Council Digest, June 1994.*

[43]     *The Difference Between a Vapour Barrier and an Air Barrier, by R.L. Quirouette, Building Practice Note No. 54, DBR/NRCC, July 1985*

[44]     *An Air Barrier for the Building Envelope, Building Science Insight, National Research Council of Canada, 1986*

[45]     *Testing of Air Barrier Systems for Wood Framed Walls, by W.C. Brown and G.F. Poirier, Institute for Research in Construction, for Canada Mortgage and Housing Corporation, June 3, 1988.*

[46]     *Air Permeance of Building Materials, by Air-Ins Inc., for Canada Mortgage and Housing Corporation, June 17, 1988.*

[47]     *Criteria for the Air Leakage Characteristics of Building Envelopes: Final Report, by Trow Inc., for Canada Mortgage and Housing Corporation, December 1989.*

[48]     *Establishing the Protocol for Measuring Air Leakage in High-Rise Apartment Buildings, by C.Y. Shaw and R.J. Magee, for Canada Mortgage and Housing Corporation, April 1990.*

[49]     *The Development of Test Procedures and Methods to Evaluate Air Barrier Membranes for Masonry Walls, by G. Hildebrand, Building Performance Centre, ORTECH International, for Canada Mortgage and Housing Corporation, November 2, 1990.*

[50]     *CMHC Research Project Testing of Air Barriers Construction Details, by Morrison Hershfield Limited, for Canada Mortgage and Housing Corporation, August 26, 1991.*

[51]     *Airtightness Test on Components Used to Join Different or Similar Materials of the Building Envelope, by Air-Ins Inc., for Canada Mortgage and Housing Corporation, September 27, 1991.*

[52]     *Practical Guidelines for Designers, Contractors and Developers on the Installation of Air Leakage Control Measures in New and Existing High-Rise Commercial Buildings, by Canam Building Envelope Specialists Inc., for Public Works Canada, No Date.*

[53]     *Some Topics on Wall Stud Design, by T. Pekoz, for the American Iron and Steel Institute, Cornell University Interim Research Report, February 1994.*

[54]     *CSSBI S5 - Guide Specification for Wind Bearing Steel Studs, Canadian Sheet Steel Building Institute, December 1990.*

[55]     *CSA S478 - 1995 Guideline on Durability in Buildings, Canadian Standards Association*

[56]     *CAN3-A370-M84 Connectors for Masonry, Canadian Standards Association*

# *Chapter 4 – Is it Good Enough?*

This chapter attempts to answer the question "Is it Good Enough? by presenting a number of analysis tools to assist the investigator along with some suggested decision making criteria. Note that this chapter should be read in conjunction with Chapter 3 where the basic behaviour of various wall elements is discussed. An emphasis is placed on structural concerns since historically, it is structural issues that have the largest economic impact on the rehabilitation of problem walls.

In addition, some direction is offered on the appropriate course of action where a problem wall is not good enough. Chapter 5 deals with remedial methodologies in more detail.

## 4.1  Warranties

If a building is under warranty, then the criteria for "is it good enough" are more related to code and contractual issues than to building performance.

As discussed in Chapter 1, the Technical Audit approach to field investigation is the most appropriate model where warranties are in effect. The investigator reviews the MV/SS wall in the context of a performance checklist (see Appendix C) and establishes a list of deficiencies. A wall under warranty is "good enough" if both of the following conditions are met:

i) The Owners got what they paid for.

ii) The wall system meets minimum code requirements.

The first criterion applies where the owners have a wall system designed and built to standards that are in excess of code minimums. If any deficiencies are uncovered then upgrading should be undertaken to meet the higher standards implied in the original design. If you purchase a Cadillac, then you are entitled to a Cadillac.

The second criterion applies in all other circumstances. If a wall has generally been designed and built to code minimums, then any deficiencies that are uncovered should be upgraded to that same standard – but not higher. If you purchase a Volkswagen, you are not entitled to a Cadillac.

A more contentious situation arises when the wall has been designed and/or built to standards that do not meet code minimums. In these circumstances, the owners are still entitled to a wall that meets code minimums even though this may represent an upgraded wall that they did not pay for in the first place.

## 4.2 Structural Evaluation by Analysis and Testing

### 4.2.1 Recommended Building Code for Evaluation

Commentary N* to the 1995 National Building Code[1] states that existing walls should be checked for either:

- The code that was in effect at the time of the original design/construction, or
- The current code using the reduced load factors in Commentary N (see Item 4.2.2).

If either of these code checks are satisfied, then the life safety requirements for the wall system have been satisfied. There are some restrictions, however, on the use of previous codes. The previous code alternative would not apply where:

- The wall system has been altered in such a way that its structural behaviour is altered.
- Significant damage, distress or deterioration is in evidence.
- Seismic is a design consideration.**
- The wall system was designed/constructed before the benchmark version of the code was in place.

The benchmark version with respect to loads is defined in Commentary N as "the earliest version which satisfies the life-safety intent of the current requirement". For wind loads, this requirement is satisfied for design/construction on or after 1960. It is unlikely that any stud walls in Canada will pre-date 1960, therefore, MV/SS walls can be deemed to be exempt from this benchmark limitation where wind loads are the governing strength consideration.

Note that Commentary N also allows the evaluation of buildings on the basis of satisfactory past performance (except for earthquake) but the wall must be at least thirty years old. It is unlikely that any MV/SS walls will qualify.

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\*    *References to NBC/95 Commentary N are based on a pre-publication "final draft" of the Commentary. Readers should confirm the accuracy of the information in this report when the published version of Commentary N is available.*

\*\*   *Seismic review of existing buildings are covered in NRC Guidelines for the Seismic Evaluation of Existing Buildings[2]. Essentially, a reduced load factor of 0.6 is proposed for earthquakes. Note that NBC/95[3] includes lower $S_p$ values for those portions of the brick tie that behave in a ductile fashion.*

### 4.2.2 Structural Evaluation on the Basis of the Reduced Load Factors In NBC/95 – Commentary N[1]

Under the guidelines proposed in Commentary N, a reduction in load factors is possible when evaluating and upgrading existing buildings. The approach is based on the philosophy that the requirement for life safety (as distinct from structural safety) should be generally equivalent to that required in the National Building Code, Part 4. (This requirement for life safety is based on an acceptable maximum annual probability of death or serious injury as a result of structural failure in a building.)

The reduced load factors are presented in the following Tables 4 –1, 4 – 2, 4 – 3 and 4 – 4, all taken directly from Commentary N. Items not pertinent to MV/SS walls have been deleted. Refer to Commentary N and a background paper by Allen[4] for more detail. See also the example calculation at the end of this section.

| *TABLE 4 - 1* *Load Factors for Structural Evaluation*[1] | | | | |
|---|---|---|---|---|
| Reliability Level[2] | Load Factor | | | Load Combination Factor $\Psi$ |
| | Dead $\alpha_D$ | Variable $\alpha_L$ or $\alpha_W^{(5)}$ | Earthquake[3] $\alpha_E^{(5)}$ | |
| 5 | 1.25 (0.85)[4] | 1.50 | 0.6 | 0.70 |
| 4 | 1.20 (0.88)[4] | 1.40 | 0.6 | 0.70 |
| 3 | 1.15 (0.91)[4] | 1.30 | 0.6 | 0.75 |
| 2 | 1.11 (0.93)[4] | 1.20 | 0.6 | 0.75 |
| 1 or 0 | 1.08 (0.95)[4] | 1.10 | 0.6 | 0.80 |

*Notes to Table 4 - 1*

[1]    *This table does not apply to post-disaster buildings.*

[2]    *Reliability = sum of the 3 indices for system behaviour, risk category and past performance in Table 4 - 2*

[3]    *See Commentary N[1] and NRC Guidelines for the Seismic Evaluation of Existing Buildings[2] for more specific guidance on the load factor for earthquakes.*

[4]    *The value in brackets applies when dead load resists failure.*

[5]    *Defined as $\alpha_Q$ in NBC/90.*

| TABLE 4 - 2 Indices for the calculation of Reliability Level | |
|---|---|
| Items to be Considered for Reliability Level | Index |
| **System Behaviour**<br>Failure leads to collapse, likely to impact people<br>Failure unlikely to lead to collapse, or unlikely to impact people<br>Failure local only, unlikely to impact people | <br>2<br>1<br>0 |
| **Risk Category (See Table 4–3)**<br>High<br>Medium<br>Low | <br>2<br>1<br>0 |
| **Past Performance**<br>No record of satisfactory past performance<br>Satisfactory past performance[1] or dead load measured[2] | <br>1<br>0 |

*Notes to Table 4 - 2*

[1]     *At least 20 years, no significant deterioration.*

[2]     *Apply to dead load factor only.*

| Table 4 - 3 Risk Category[1] | |
|---|---|
| **Category** | **Description** |
| High | Schools and other occupancies where many people are likely to be exposed to risk associated with the failure ($N^{(2)}$ = 100 or more), buildings of major heritage importance, or industrial or other facilities with hazardous occupancies. |
| Medium | Other occupancies where fewer people are likely to be exposed to risk associated with the failure ($N^{(2)}$ = 5 to 100). |
| Low | Other occupancies where the floor area or adjacent outside area exposed to the failure is not likely to be occupied by people and, when occupied, by a small number of people only ($N^{(2)}$ < 5). |

*Notes to Table 4 - 3*

*(1)*     *This table does not apply to post-disaster buildings*

*(2)*     *The estimated maximum number of people exposed to risk associated with the failure, N, may be estimated as follows:*

*N = [Occupied area exposed to risk, in m² [5]] x [occupancy density[3] [5]] x [duration factor[3] [4] [5]]*

*(3)   For building occupants, this parameter may be estimated using Table 4 - 4.*

*(4)   Duration factor = [average weekly hours of human occupancy]/100 ≤ 1.0*

*(5)   For people outside, adjacent to the building, these parameters should be assessed approximately, using the same concepts as for building occupants.*

| Table 4 - 4 Parameters for the Estimation of N | | |
|---|---|---|
| Primary Use | Occupancy Density Persons per m² | Average Weekly Hours of Human Occupancy |
| Assembly | 1.0 | 5 – 50 |
| Mercantile and personal services | 0.2 | 50 – 80 |
| Offices, institutional, manufacturing | 0.1 | 50 – 60 |
| Residential | 0.05 | 100 |
| Storage | 0.01 to 0.02 | 100 |

### 4.2.2.1   Example Calculation

The reduced load factors in Table 4 – 1 are intended to apply to the specific component addressed by the calculation. For purposes of this example, the steel stud is being checked on the assumption that the brick veneer does no structural work and all the wind load is carried by the back-up. This is the design procedure required by the code[5] for new buildings.

Assume a 10 year old high-rise residential building 19 m tall, 21 m wide and 46 m long. The MV/SS wall under review is located on the 46 m rear elevation of the building. If the MV/SS wall collapses, those exposed to the highest risk of death of injury are assumed to be in the adjacent ground area. The following parameters might be appropriate:

*From Table 4 - 4*

Occupancy Density (Residential)                    = 0.05 persons/m²
Average Weekly Hours of Human Occupancy            = 100

Note that a higher occupancy density might be appropriate for other areas of the building, for example, near the front entrance. The average hours of weekly occupancy might also require adjustment since the rear of the building might be only infrequently accessed for garbage collection or more frequently for parking or children playing. The type of loading is

also relevant here. People are less likely to occupy adjacent ground areas during a wind storm since they seek shelter inside the building but might be more likely to occupy the same space during an earthquake since there is an inclination to exit the building during a seismic event.

*From Table 4 - 3*

Duration Factor = 100/100 = 1.0

Assume the area exposed to falling brick extends out 10 m from the face of the building and continues for 10% of the length of the wall near each corner (20% of 46 m total). This dimension coincides with the location of maximum wind load as defined in Figure B–8 NBC/90 Supplement[6].

Therefore, N = (0.2 x 46 x 10) x (.05) x (1.0) = 4.6 and
Risk Category = Low

*From Table 4 - 2*

Past Performance Index = 1 (no record of satisfactory past performance since building age < 20 years)

Risk Category Index = 0 (Low)

System Behaviour Index = 1 (Failure unlikely to lead to collapse, or unlikely to impact people)

The System Behaviour Index is intended to reflect the impact of the component failure. What will happen if the component (i.e. the stud in this case) fails? Are there protective features of the structural system (such as alternative load paths) that, given structural failure, reduce the likelihood of people being injured or killed? Considerable engineering judgement is required. For the case of a stud failure, alternative load paths exist in that the brick veneer is capable of acting as a structural member spanning horizontally past any failed studs. This horizontal bending behaviour can also add strength at building corners where the brick is supported three sides (top, bottom and the building corner). In addition, laboratory evidence[7,8] indicates that MV/SS walls have substantial strength reserve even where the boundary conditions would not allow the brick to span horizontally.

*From Table 4 - 1*

Reliability Level = Sum of the 3 indices from Table 4 – 2 = 1 + 0 + 1 = 2

and $\alpha_w$ = 1.2 (as opposed to 1.5 for new buildings – a 20% reduction in safety factor).

#### 4.2.2.2 *Loads Recommended for Use In Evaluations*

These recommendations have been taken from NBC/95 Commentary N[1].

### *Loads Due to Movement (Temperature, Moisture, Elastic and Creep Differential Deflections)*

Commentary N suggests that in the absence of any symptoms of distress, loads due to movement can be ignored in the structural evaluation of an existing building. Past experience with the building will be the best indicator of problems in this regard. The number of years of experience that are necessary will vary depending on the type of movement being observed. The time frames for various types of movement were discussed in Chapter 3.

### *Loads Due to Wind*

It is recommended that the wind loads specified in the current building code be used. Some reduction (or increase) might be justified if wind speeds at the site are monitored and compared with the local weather station recordings. Future changes in the local topography or changes in the building shape might, however, invalidate this type of study.

### *Loads Due to Earthquake*

The loads due to earthquake should be taken from NBC/95 in combination with the 0.6 reduced load factor in Table 4 – 1. This reduced factored earthquake load should only be considered suitable as a "triggering criterion for seismic upgrading .... for the design of the upgrading, the load factor should be increased preferably to the NBC value." [1]

#### 4.2.2.3 *Factored Resistances for Use In Evaluations*

Use the resistances (including the appropriate resistance factors) from the relevant material standards[1].

### 4.2.3 Structural Evaluation Using Conventional Analysis

Building codes have historically required that all of the wind load be transferred to the steel stud back-up and the structural contribution of the veneer be ignored. This analytical approach may ignore some considerable strength reserve in the system but has the merit of simplicity and is a suitable "first pass" method of analysis. This approach is outlined in considerable detail in the CSSBI Lightweight Steel Framing Manual[37]. Refer to S304.1[27] for maximum loads on ties.

Conventional analysis should be combined with the reduced load factor approach outlined in Item 4.2.2.

### 4.2.4 Structural Evaluation on the Basis of Finite Element Analysis

Currently available analytical tools contain too many simplifying assumptions to accurately model the real behaviour of MV/SS wall systems. These simplifying assumptions (see Item 4.2.3) typically result in conservative designs that are economical and practical to implement in a new building but may impose burdensome and unnecessary repair costs once the structure is complete.

To address this problem, a finite element program (named MVSS[9]) has been developed as an aid for the structural evaluation of problem walls*. This program is intended to provide engineers with a sophisticated and accurate analysis tool that is also economical to use. The program includes the following capabilities:

- Two way bending of the brick is considered. The program treats the veneer as a flexural plate with different properties for strength and stiffness in the horizontal and vertical directions.
- The flexibility of the steel components (ties, tracks and studs) are modelled.
- Maximum tie strengths can be specified. As the program cycles through various load stages, ties that reach their ultimate strength are assumed to fail and are subsequently discounted.
- The effect of windows and other openings can be included in the analysis.
- Loads from windows or doors can be distributed to the stud frame or the veneer using pre-selected attachment points.
- Various boundary conditions for the brick and the stud (such as corners and intersecting shearwalls) can be specified.
- An iterative analysis routine is used to predict the cracking behaviour of the wall. The program calculates both the configuration of the cracking and the loads at which various stages of cracking occur (for example initiation of the 1st crack, full propagation of the 1st crack, initiation of the 2nd crack, etc.)
- Alternatively, the user can specify the cracking configuration.

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\*    *The basic analysis routine for the finite element program was developed as part of another CMHC project[10] and then customized with a number of features specifically for this project. The program and worked examples are distributed on 3 - 1.44 megabyte diskettes and are accompanied by a separate (131 pages) User Reference Manual[9].*

- The effect of missing brick ties or the introduction of retrofit brick ties can be studied.
- The structural effects of full or partial pressure equalization can be studied.

A number of "user–friendly" features are included:

- The program contains a finite element mesh generator which relieves the user of a lot of tedious input while at the same time retaining the flexibility to handle a wide variety of wall geometries.
- Data banks have been included with experimentally derived stiffnesses for ties, top and bottom tracks (based on the McMaster studies[11, 12]) and stud properties taken from product literature.
- The data banks can be added to or amended as required.
- Extensive use has been made of pop–up screens to facilitate data entry.
- Plotting routines have been added to allow the user to see on–screen the input geometries and boundary conditions, output stress contour diagrams, tie forces, deflected shapes and crack patterns.
- The graphical display can be dumped to either a file or a printer (only Postscript and HP–GL plotters are supported).
- Detailed output data is sent to an ASCII file for easy access by any standard DOS editor or word processor. (These output files can be dumped to any printer type.) These files contain output data for stud deflections, moment and shear, for tie loads, and for veneer stresses. In addition, the input geometry generated by the program is provided.

Refer to the separate User Reference Manual that accompanies the program for detailed description of the program capabilities and limitations. The Manual also contains complete instructions on how to use the program along with 4 case studies that were chosen to illustrate the full capabilities of the program.

### Case Study 1 – A

This case study corresponds to a wall tested at McMaster University, 5.2 m long with 2.8 m high veneer. (For a layout drawing, see the separate User Reference Manual Page 49.) This example serves an introduction to the program, illustrates how to handle irregular stud spacings and how to take advantage of centreline symmetry.

### Case Study 1 – B

In Case Study 1 – A the program generates the midheight crack. This case study is identical except that the midheight crack is forced.

### Case Study 2 – A

This case study was selected to demonstrate a number of program capabilities. (For a layout drawing, see the separate User Reference Manual Pages 88 – 89.) The selected wall includes 2 window openings, missing ties, jamb studs not

connected together and an intersecting shearwall. The last stud is connected to the shearwall but the brick veneer is not. Centreline symmetry is used to halve the input. Note that this example is about the largest possible problem that the program is capable of handling without exceeding RAM limits.

### Case Study 2 – B

Case Study 2 – B is identical to 2 – A except that the original ties have been replaced by retrofit ties. (For a layout drawing of the ties, see the separate User Reference Manual Page 112.) The retrofit ties have been attached to both the inner and the outer jamb studs to insure that they work together to resist the wind loads applied from the window area.

### Helpful Notes (See Also Appendix D)

* Most structural analysis programs accept a design load as input and generate the response of the structure at that load. This program, instead, calculates the load capacity of the wall at various stages of cracking in the wall.

  The load and the resulting response of the wall are first calculated at the initiation of cracking in the veneer. The user can then direct the program to re-analyzing the wall any number of times to allow that crack to propagate (with ever increasing loads) element by element until it is fully developed. This process can be continued for the initiation of the second crack and its propagation. Typically, the cycling and re-analysis is stopped once the design load is reached. Note that the behaviour of the wall system is linear only up to the initiation of the first crack.

* The output from the program is strongly dependent on the input properties for the brick veneer. These properties include modulus of elasticity parallel and perpendicular to the bed joint, and tensile strength parallel and perpendicular to the bed joint. The user may elect to use code values or to use the results from small specimen testing. See Item 4.2.5, Test Methods for Small Scale Specimens, for further guidance.

* As discussed in Item 3.1, MV/SS walls should be checked for the pre–cracked and the post–cracked condition. Typically (although not always – particularly with more complicated geometries) , the pre–cracked condition gives the maximum tie loads and the post–cracked condition gives the maximum load on the steel stud back–up. It may be prudent to run a particular wall geometry with the maximum and minimum expected brick strengths and stiffnesses. The maximum brick strength, for example, would delay cracking and likely produce the maximum tie load. Similarly, the minimum brick strength would likely produce the maximum load on the stud.

- It is recommended that full load factors from NBC/95, Part 4, in combination with the resistance factors from the appropriate material standard be used with the output from this program.

### 4.2.5 Full Scale In-Situ Testing*

Full scale testing is typically undertaken to verify the load carrying capacity for the as-built condition.

One possible test apparatus is illustrated in Figure 4 – 1 and 4 –2. See also ASTM E72[13] for additional information.

---

Pipe and threaded rod
with adhesive anchor to slab
(top and bottom)

Slab

Threaded rod

Stud

Brick veneer

Spreader beam location
to simulate positive
wind loads

Jack and load cell

Spreader beam location
to simulate negative
wind loads

Spreader beam through
bolted to brick

Structural timbers sized
for load requirements

FIGURE 4 - 1     SECTION THROUGH FULL SCALE IN-SITU
                 TEST APPARATUS (See also Fig. 4 - 2)

Note: This apparatus is a variation on a rig that was used successfully by
J. Stuart Hall & Associates Ltd. on a building in the Ottawa area

Soft joint beneath shelf angle

Slab (shelf angle not shown)

Spreader beams

Control joint through brick and backup on either side of test panel

Bolt - through beam and brick veneer

Steel stud

Jack and load cell

Threaded rod
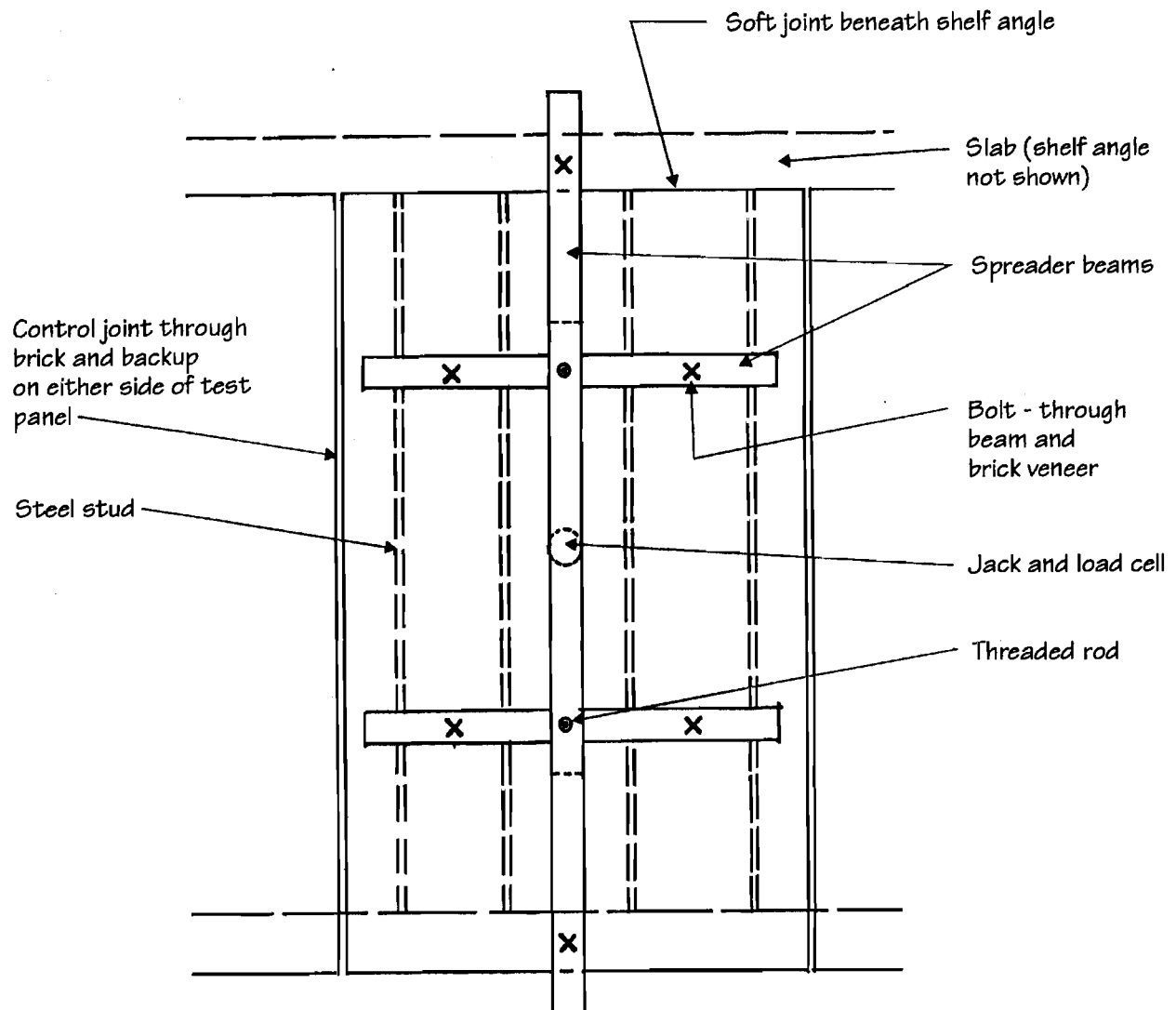
FIGURE 4 - 2     ELEVATION - FULL SCALE IN-SITU
TEST APPARATUS (See aslo Fig. 4 - 1)

### *Full Scale Testing – Discussion*

- Since the brick veneer has substantial horizontal bending strength and stiffness, it is necessary to cut control joints as illustrated in Figure 4 – 2 in order to isolate the portion of veneer that is responding to the applied load.

- The steel stud back–up system also has some ability to span horizontally. The control joint shown in Figure 4 – 2 should continue through the inner and outer sheathings on the back–up and through the through–the–knockout style bridging.

- If the strength of the connection between the top and bottom track and the concrete is in doubt, the tracks should be cut at the location of the control joint. Similarly, if the strength of the outer top track is in doubt, the inner and the outer top track should be cut at the location of the control joint. Note that it may be difficult to isolate portions of top and bottom track with concrete fasteners appropriately placed.

- On some projects (walls with full height windows for example), isolated walls occur without the need to introduce control jointing. However, these isolated walls are frequently bounded by windows on either side and care must be taken to insure that the wall is disengaged from the structural elements surrounding the window.

- It can be difficult to take advantage of the two way plate behaviour of the brick. A test apparatus such as the one illustrated in Figures 4–1 and 4–2 is not likely appropriate in these circumstances and air bag type loading might be required.

- As discussed in Item 3.1.1, MV/SS walls exhibit different behaviour before and after cracking. Before cracking the brick carries 80 – 90% of the load and the top tie is the most heavily loaded. After cracking most of the load is transferred to the steel stud back–up and the midheight tie is the most heavily loaded. Testing should examine, if possible, both the pre and post–cracked conditions. Obviously, if the wall has already cracked then this is the only condition that can be checked.

- On projects missing a soft joint under the shelf angle, a soft joint should be introduced into the portion to be tested. Removing the course of brick immediately below the shelf angle will suffice. On projects with a soft joint, the caulking and backer rod should be removed. On tests at McMaster University[7], it was found that the caulking and backer rod could (but not in any reliable way) transfer substantial load and thus reduce the load on the top brick tie.

- On walls where a hard joint beneath the shelf angle has been in place for the life of the structure, it is very likely that the brick veneer has been subject to substantial compressive stresses. These stresses typically strengthen and

stiffen the brick veneer such that the steel stud backup experiences very little load.

On the test wall once a soft joint is introduced, the steel studs might experience load for the first time and for the first few load cycles, the strength and stiffness of the studs will be enhanced by the composite action of the drywall sheathings[14, 8].

In these circumstances, it is necessary to condition the test wall with cyclic loading prior to the proof load test.

- The typical wall stud (illustrated in Figure 4 – 2) may not be the critical design condition. On many projects, the trimmers around openings (built-up jamb studs and head and sill track members) and the ties around openings are the most heavily loaded elements in the MV/SS wall. Care is required to insure that the behaviour of these areas of the MV/SS wall is accurately reflected by the test procedure.

- Before testing, a number of inspection openings should be made in order to find the worst case conditions in the wall. It is unlikely that the worst case conditions for all the critical wall elements will occur at one spot and a number of test locations will, therefore, be required.

The selection of the worst case condition may not be obvious. For example, where the strength of brick ties is a concern a test wall with a large air gap free of mortar bridges should be selected. The large air gap gives the minimum compressive strength for the ties, likely the least embedment of the tie in the mortar joint and the fewest mortar bridges spanning the air gap which can act as alternative load paths when the ties are in compression.

- It can be difficult to determine how many tests are necessary to have a statistical representation of the behaviour of the entire MV/SS wall.

If analytical techniques indicate that the wall is substantially overstressed, a larger test sample is warranted. In this case, the investigator is entirely dependent on full scale testing to establish the structural integrity of the wall. Every construction detail (structural and architectural) can be important and more testing is necessary in order to cover possible variations.

If a large number of inspection openings have been made and the worst case conditions selected, then the number of tests can be reduced.

If a number of tests have been run and there is a large amount of statistical scatter, then a larger test sample would be appropriate particularly when the average of the test results is close to the target load.

- Where there is evidence of deterioration in the wall assembly such as corrosion of the steel parts, then periodic re-testing may be required to monitor the

deterioration to be certain that the structural integrity of the wall has not degraded below acceptable levels.

- More will be learned from the load testing if the various components of the wall assembly are observed before, during and after the test. To facilitate viewing, inspection ports can be cut in the interior and exterior sheathing. (Complete removal of the sheathing would adversely affect the torsional restraint and perhaps the ultimate strength of the steel stud backup.)

- The test rig illustrated in Figures 4 – 1 and 4 – 2 applies all of the lateral load to the brick veneer and none to the steel stud backup (except for the loads transferred by the brick ties). For the case of wind, this type of loading is equivalent to saying that there is no pressure equalization whatsoever. If pressure equalization does occur (full or partial) in the wall, the observed test behaviour may not be entirely representative. Also some structural elements such as air barriers and their attachment details will not be tested.

### Loads and Safety Factors

There are 2 types of testing – proof tests to some pre-determined load and ultimate tests where the assembly is loaded to failure. An unsuccessful proof load tests is, in effect, an ultimate test.

Proof load tests are the most economical and all that is necessary to establish structural safety. A proof load test is unsuccessful if the assembly fails (reaches its ultimate strength) or if the assembly deflects in an uncontrollable fashion (large plastic deformations with no increase in load carrying capacity) before reaching the target load.

To establish the target load, it is suggested that the specified lateral loads from NBC/95[3] (both positive and negative) be combined with the 1.6 suggested load factor for testing from NBC/95 Commentary N. The commentary also suggests that the load be sustained for a period of 24 hours. As discussed previously, it may also be necessary to condition the assembly prior to proof load testing with cyclic loading.

Serviceability should be checked at specified load levels with no factor of safety.

### Full Scale Testing – Advantages

- The real performance of the wall is measured with no simplifying assumptions about structural behaviour or material properties.

### Full Scale Testing – Disadvantages

- Testing can be disruptive and costly.
- It can be difficult to isolate a portion of the wall so that no neighbouring wall elements are accidentally participating in resisting the applied load.

- It can be difficult to simulate loading on the back-up for walls where pressure equalization occurs.
- It can be difficult to choose a statistically representative test sample.
- Care is required to insure that loading will produce the maximum effects for various elements in the wall.

### 4.2.6 Test Methods for Small Scale Specimens

Small scale tests provide useful input for various engineered masonry approaches including the MVSS finite element program discussed previously.

The test methods discussed here provide flexural bond strengths for tension parallel to and perpendicular to the bed joints. The prisms used for testing are assumed to be cut from the in-situ wall.

Note that for all of the proposed procedures, tests should be undertaken with the tooled joint in tension and repeated with the tooled joint in compression.

#### Flexural Bond (Tensile) Strength Perpendicular to the Bed Joint

Flexural bond strengths can be determined by the bond wrench method, ASTM C 1072[15] or by the beam test method, ASTM E 518[16].

The beam test is intended to be used with a minimum 450 mm high stack bond prism of bricks and tested with 3rd point loading or uniform loading by means of an air bag. A running bond specimen can substitute for the stack bond prism specified in the standard.

The bond wrench test attaches a lever arm and clamping device to the top course of brick in the specimen and fails the bond by applying a moment (and an axial load which has little effect on the test results). Lever arms up to 1200 mm in length have been used in the field. This test has the advantage of providing failure data for every joint in the test specimen as opposed to only one joint from the beam test. See also Item 2.2.

Refer to Drysdale et al.[17] for more information.

#### Flexural Bond Strength Parallel to the Bed Joint

Specimens can be tested by the bond wrench method or the beam test method discussed previously. A minimum specimen size of 4 units long and an even number of courses (say 4 courses) high is recommended. Half units in alternate courses at the end of the specimen will insure symmetry[17].

Refer to Drysdale et al.[17] for more information.

### Elastic Modulus

The elastic modulus cannot be determined from the small flexural specimens discussed above. Deflections at failure are too small to yield useful results. The investigator can develop an elastic modulus (assumed to be the same in the horizontal and the vertical directions) using one of the following two methods:

- Use a modified S304[5] modulus.

    $E_m = kf_{m'}$ , where k = 500 to 600 for North American brick[17].

- Do compression testing on a prism cut from the wall. Drysdale et al.[17] recommend a specimen 1 unit long and perhaps 6 units high to accommodate a 200 mm gauge length. The gauge length should include an even number of units and mortar joints. Refer to ASTM E 447[18] and Drysdale et al.[17] for more information.


## 4.3   Is It Good Enough? – Decision Making Criteria for Strength, Serviceability and Building Science Issues

This section provides accept/reject criteria for various distressed elements of the wall system. Refer to the parallel discussion in Chapter 3 where the building science, serviceability and strength behaviour of these elements was discussed.

For structural strength considerations, the analytical procedures suggested in Item 4.2 should be used to derive the factored loads and stresses on individual elements in the wall system. These factored loads and stresses should be checked against the decision making criteria in this section. These criteria are in effect factored resistances for the various wall elements.

For serviceability, the results of the analytical procedures in Item 4.2 should again be used but this time at specified load levels to be checked against the suggested deflection limits in this section.

Building science issues are dealt with in a global fashion (Item 4.3.1) and are not discussed for each individual element of the wall system.

Information can be located in this section by using the index in Table 4 –1 which directs the reader to the appropriate item discussed in the text. The table lists alphabetically:
- Symptoms observed, and
- Wall element under distress

| TABLE 4 - 1 *Index - Refer to Item Numbers Indicated* | | | |
| --- | --- | --- | --- |
| Wall Component or Symptom | Strength | Serviceability | Building Science |
| Air barrier | *See 4.3.2.7* | | *See 4.3.1* |
| Air leakage | *See air barrier* | *See air barrier* | *See air barrier* |
| Air space | | | *See 4.3.1* |
| Brick | *See 4.3.2.4(a) & (b)* | *See 4.3.2.4(a)* | *See 4.3.1* |
| Bridging | *See 4.3.2.11* | | *See 4.3.1* |
| Caulking | *See sealants* | *See sealants* | *See sealants* |
| Cavity | *See air space* | *See air space* | *See air space* |
| Control joints | *See cracking* | *See cracking* | *See cracking* |
| Corrosion | *See 4.3.2.18* | | *See 4.3.1* |
| Cracking – flexural | | *See 4.3.2.10* | *See 4.3.1* |
| Cracking – horizontal movement | | *See 4.3.2.3(a) & (c)* | *See 4.3.1* |
| Cracking – vertical movement | | *See 4.3.2.3(b) & (c)* | *See 4.3.1* |
| Deflection | | *See 4.3.2.10* | *See 4.3.1* |
| Drywall – exterior | *See sheathing - exterior* | *See sheathing - exterior* | *See sheathing - exterior* |
| Drywall – interior | *See sheathing - interior* | *See sheathing - interior* | *See sheathing - interior* |
| Drying | | | *See 4.3.1* |
| Dust shadowing | | | *See 4.3.1* |
| Efflorescence | | | *See 4.3.1* |
| Expansion joints | *See cracking* | *See cracking* | *See cracking* |
| Flashings | | | *See 4.3.1* |
| Freezing and Thawing | | | *See 4.3.1* |
| Head track | *See window* | *See window* | *See window* |
| Insulation – batt | | | *See 4.3.1* |
| Insulation – exterior | *See 4.3.2.6* | | *See 4.3.1* |
| Lintels | *See 4.3.2.2* | *See 4.3.2.2* | *See 4.3.1* |
| Moisture | | | *See 4.3.1* |
| Moisture movements | | *See 4.3.2.3(a) & (b)* | *See 4.3.1* |
| Mortar and mortar joints | *See 4.3.2.4(c)* | *See 4.3.2.4(c)* | *See 4.3.1* |
| MV/SS walls as a system | *See 4.2* | *See 4.2* | *See 4.3.1* |

| TABLE 4 - 1 *Index - Refer to Item Numbers Indicated* | | | |
|---|---|---|---|
| **Wall Component or Symptom** | **Strength** | **Serviceability** | **Building Science** |
| Movement | *See cracking, top track* | *See cracking, top track* | *See cracking, top track* |
| Parapets | *See 4.3.2.16* | | *See 4.3.1* |
| Rain screen | *See 4.3.2.7* | | *See 4.3.1* |
| Sealants | | *See 4.3.2.17* | *See 4.3.1* |
| Sheathing – exterior | *See 4.3.2.6* | | *See 4.3.1* |
| Sheathing – interior | *See 4.3.2.8* | | *See 4.3.1* |
| Shelf angles | *See 4.3.2.1* | *See 4.3.2.1* | *See 4.3.1* |
| Sill track | *See window* | *See window* | *See window* |
| Spalling | | | *See also freezing and thawing, efflorescence* |
| Studs – built-up at jamb | *See 4.3.2.14* | *See 4.3.2.14* | *See 4.3.1* |
| Studs – typical | *See 4.3.2.9* | *See 4.3.2.10* | *See 4.3.1* |
| Thermal bridging | | | *See 4.3.1* |
| Thermal expansion and contraction | | *See 4.3.2.3(a), (b) & (c)* | *See 4.3.1* |
| Thermal – R value | | | *See 4.3.1* |
| Ties | *See 4.3.2.5* | *See 4.3.2.5* | *See 4.3.1* |
| Track – bottom | *See 4.3.2.12* | *See 4.3.2.12* | *See 4.3.1* |
| Track – top assembly | *See 4.3.2.13* | *See 4.3.2.13* | *See 4.3.1* |
| Vapour retarder | | | *See 4.3.1* |
| Vents and weepers | | | *See 4.3.1* |
| Weepers | *See vents* | *See vents* | *See vents* |
| Window – head and sill members | *See 4.3.2.15* | *See 4.3.2.15* | *See 4.3.1* |

## 4.3.1  Building Science Issues – Is It Good Enough?

In the context of this report, building science issues include the transmission of air, vapour, moisture, and heat. As was discussed in Chapter 1, the symptom driven approach is sufficient for the review of these issues. Building science deficiencies are best evaluated on the basis of satisfactory past performance.

### 4.3.1.1  *No Symptoms of Distress*

If there are no symptoms, then the performance is good enough.

A number of conditions should be imposed on acceptance by this approach:

- Careful examination of the building by an experienced investigator does not expose any evidence of distress. See Chapter 3 for the types of distress that might be observed with respect to building science issues. See Chapter 2 for methods of gathering field information.

- The building has sufficient performance history. This will vary depending on the type of distress being observed. For example, corrosion of steel parts might take several years to develop but the moisture that is causing the corrosion might be observed in the first winter if the building is opened up at the appropriate time.

- There have been no recent changes to alter the building science performance of the building. For example, the air handling system may have been revised in the interests of energy efficiency. Perhaps there are fewer air changes and interior relative humidities rise. Imperfections in the air barrier which previously were of no consequence now result in substantial condensation in the stud space due to the exfiltration of higher humidity air during the winter months. See Chapter 3 for the types of distress that might result.

While a number of objective analytical criteria have been developed to assess building science performance – only very rarely should these criteria replace the evaluation based on satisfactory past performance. The building is, in effect, a full scale building science experiment. The best predictor of future performance is an extrapolation of past performance.*

For buildings with insufficient performance history or for buildings with recent changes that may affect its performance, the best course of action is periodic monitoring of the wall to determine if symptoms occur.

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\*   *As discussed in Chapter 1, this approach is acceptable for building science but not structural issues. Structural loads are relatively infrequent and the consequences of structural failure more severe in terms of public safety.*

### 4.3.1.2 *Some Symptoms of Distress*

Where symptoms of building science distress are observed one or more of the following actions may be appropriate:

#### *Do Nothing:*

Some symptoms of building science distress can be tolerated provided the following conditions are satisfied:

* The rate of deterioration of building components is acceptable. The rate of deterioration can be determined from past performance or by monitoring future performance. An acceptable rate of deterioration will depend to a large extent on the expected future service life for the wall*.
* The inconvenience to occupants is acceptable. For example, some leakage problems might be quite benign in terms of deterioration of components but extremely annoying to building occupants.
* The aesthetics of the building are not significantly affected. For example, some efflorescence might pass the previous two tests but be too unsightly.

#### *Maintain or Restore*

The maintain or restore options are primarily economic decisions. In simple terms, is it worth spending money on the wall system in the context of the expected future service life? See Chapter 5 for further discussion.

Note that periodically monitoring the wall after the remedial work is complete is a useful technique to check the viability of a repair procedure – has the rate of deterioration slowed down sufficiently?

#### *Re-build*

This option should be considered as a last resort because of the inherent expense and disruption to occupants. Building science issues may trigger the need to re-build a problem wall where the structural components of the wall have deteriorated to the point where the wall poses a threat to public safety. For example, moisture problems in the wall might lead to excessive corrosion of the steel parts.

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\*    *CSA S478 - Guideline on Durability in Buildings[20] defines service life as the actual period of time during which the building or any of its components performs without unforeseen costs or disruptions for maintenance and repair.*

## 4.3.2  Strength and Serviceability Issues – Is It Good Enough?

In the context of this report, structural serviceability means the control of excessive deflection of steel components or excessive cracking of the veneer. The cracking of the veneer can in turn be related to movements in the veneer, the building structure or to the inherent flexibility of the steel stud backup. See Chapter 3 for more detail. As discussed in Chapter 3, movement in the brick veneer and differential movement between the brick and the structure can be restrained or unrestrained. Note that when restrained movement results in significant internal stresses, the serviceability label is not correct and the effect of these stresses should be combined with other loads as required by NBC/90[19] and NBC/95[3] as load effect "T".

NBC/95 Commentary N[1] states that "For existing buildings, in many cases, demonstration of satisfactory performance eliminates the need to apply the serviceability criteria in [NBC] Part 4 and referenced structural standards for evaluation." A ten year minimum performance history is suggested. The conditions for acceptance outlined in Item 4.3.1.1 should be applied if this approach is adopted. Note, however, that there are some serviceability issues where satisfactory past performance may not be a sufficient condition for acceptance particularly for wall systems that have not been subjected to a significant portion of the design load or temperature extremes.

In the context of this report, structural strength means the ability of various elements of the wall system to resist applied loads without imposing a risk to public safety through failure. Factored resistances for various wall elements are provided in the following section. In many cases, only references are given since the calculation of the strength of many components is beyond the scope of this report. The results of (or references to) recent research have been included where appropriate.

### 4.3.2.1  *Shelf Angles – Strength and Serviceability – Is It Good Enough?*

Shelf angle behaviour and typical design practices are outlined in Item 3.2. The determination of loads and resistances for shelf angles is complex requiring sophisticated analytical techniques such as finite elements, the approaches suggested by Grimm[21] or the more recent Tide and Krogstad[22].

Successful past performance is not a sufficient condition for the acceptance of in-service shelf angles. Where the soft joint under the shelf angles does not function or where the connection of the angle to the structure is inadequate, loads that should be carried by the angle may be instead carried by the brick below. Also corrosion damage to shelf angles and connections is common and may require investigation in spite of an apparently successful performance to date.

See also Item 4.3.2.18 – Corrosion

Inadequate shelf angles require reinforcement or replacement – see Chapter 5.

### 4.3.2.2 *Loose Angle Lintels – Strength and Serviceability – Is It Good Enough?*

Loose angle lintel behaviour and typical design practices are outlined in Item 3.3. The analysis of loose angle lintels is quite complex with some unknown amount of arching in the brick, bending and torsion of the angle and cantilever deflection of the angle leg. See Drysdale et al.[17] and BIA Technical Note No. 31B[23].

Successful past performance may in some instances be a sufficient condition for acceptance.

See also Item 4.3.2.18 – Corrosion.

Inadequate loose angle lintels require reinforcement or replacement.

### 4.3.2.3 *Brick Veneer Horizontal and Vertical Movement Joints – Serviceability – Is It Good Enough?*

#### a) Horizontal Movements

Horizontal movements can lead to cracking of the veneer when vertical control joints are absent or inadequate. See Item 3.4 for more detail.

This type of movement can lead to unsightly cracking and significant increases in water penetration through the brick veneer. Where movements are large enough, some brick may be dislodged from the wall posing a safety risk to people in the adjacent ground area below.

This type of cracking can be deemed "good enough" provided:

• The cracks are not too objectionable aesthetically.
• There are few or no symptoms of distress due to water penetration through the exterior veneer.
• The brick is stable and unlikely to dislodge from the wall. The integrity of the brick will depend on the size and location of the cracking and the location of brick ties.
• The wall is periodically monitored. The observed cracking may not be stable (see Item (c) ).

Walls that are not "good enough" typically require repair of the observed cracking and installation of vertical control joints. See Chapter 5.

### b) Differential Vertical Movements

Differential vertical movements between the brick veneer and the structure can led to distress in the veneer when horizontal control joints (soft joints) beneath the shelf angles are absent or ineffective. Even where soft joints are present, distress may occur at locations even where the shelf angle is discontinuous – most commonly at building corners. See Item 3.4 for more detail.

From a public safety standpoint, restrained vertical movement is typically more hazardous than horizontal movement. Large compressive stresses can develop which may result in overall buckling of the veneer and its loss from the structure. (As discussed in Chapter 3, local spalling at shelf angle locations is another common symptom.)

Estimating stresses due to restrained differential vertical movement is a complex issue. Fenton and Suter[24] have proposed a computer model that includes the following capabilities and limitations.
* reinforced concrete, steel and load bearing masonry can be modelled
* veneer can be connected to the masonry by ties alone, by ties and shelf angles at single or multiple floor spacings
* yield and stiffness properties of the ties and shelf angles are considered in the analysis
* various movement joint widths can be accommodated
* various time periods from the completion of construction of the structural frame can be specified
* maximum number of stories is 25
* all forces are assumed to be concentric (shelf angles typically exert an eccentric load on the brick which might lead to local failure or an earlier buckling load than that predicted under concentric load)
* veneer must be clay brick.

The computer generated stresses can be used in a subsequent engineering analysis to assess the probability of spalling or buckling of the veneer. See Fenton and Suter[24, 25] for more detail.

For bulging veneer another possible accept/reject criterion would be to limit the out–of–plane movement of the veneer such that tension stresses do not develop on the convex side of the veneer. Using conventional mechanics of materials, this implies a limiting out–of–straightness of t/6 for a concentric axial load. This approach is not recommended because:

* The compressive load in the veneer likely has some unknown eccentricity which would indicate tension at some out–of–straightness less than t/6. Fenton and Suter[24] pointed out possible sources of eccentricity, particularly shelf angle forces applied to the inside edge of the masonry owing to the rotation of the angle as it deforms.

- The out-of-straightness in the veneer may not be stable. See Item (c).

### c) Stability of Horizontal and Vertical Movement in Older Buildings

Heliker and Brock[26] reported two incidences of sudden failures in distressed veneer walls that had been dormant for a number of years. Although the distress and cracking in the two veneers appeared to have stabilized, they theorized that the onset of unusually hot weather caused the sudden deterioration that they observed.

They concluded that the tendency of these failures to slow, or even completely stop does not insure that they will remain dormant during unusual weather conditions, not yet experienced by the building envelope.

### 4.3.2.4 Brick and Mortar – Strength and Serviceability – Is It Good Enough?

#### a) Flexural Strength and Elastic Modulus

For conventionally designed MV/SS walls the brick and mortar are required to perform a relatively trivial structural function since the entire wind load is carried by the steel stud backup. Under these circumstances, any reasonably sound brick and mortar combination will be "good enough".

Where the evaluation is relying on the structural capability of the brick veneer, then these properties should be determined from code recommendations (S304.1[27]) or from testing as discussed in Item 4.2.6.

#### b) Overhang

Excessive brick overhang can be deemed "good enough" provided the brick will not disengage during an earthquake, bearing stresses are not excessive (S304.1[27]) and the eccentricity is acceptable given the strength of the veneer, the ties and the steel stud backup. Some reserve bearing may also be required where thermal or moisture movement of the veneer is likely to occur. Considerable engineering judgement is required in the evaluation of these issues.

If shelf angle overhang is not "good enough", the horizontal leg of the angle can be extended. See Chapter 5.

#### c) Mortar Joints

Repointing of eroded mortar joints may be required if the joints are eroded more than 3 mm from the surface of the brick for flush joint designs or can be easily scraped away with a metal tool (The Repointing Spec.[28])

### 4.3.2.5  *Brick Ties – Strength and Serviceability – Is It Good Enough?*

The load on brick ties can be derived in accordance with the require-
ments of S304.1[27] or the MVSS finite element computer program. The re-
sistance of ties is best determined by the test procedures outlined in
A370–94, Connectors for Masonry[29], from the tie manufacturer's test re-
ports, or from the results of an extensive testing program on typical
MV/SS brick ties* by Drysdale and Wilson[11]. Note that strength, stiffness
and mechanical free play are all important parameters of tie performance.

For earthquake loads, refer to NBC/95[3] for the required loading as a
function of the ductility of the tie.

### 4.3.2.6  *Exterior Insulation and/or Exterior Sheathing – Strength – Is It Good Enough?*

Exterior insulation/sheathing may behave as a deliberate or accidental air
barrier. See Item 4.3.2.7, Air Barriers.

### 4.3.2.7  *Air Barriers – Strength – Is It Good Enough?*

#### *Wind Loads on Air Barriers*

The factored loads on air barriers can be determined from the reduced
load factor approach presented in Item 4.2.2. Note that the structural fail-
ure of a typical air barrier has almost no consequences with respect to life
safety, but if left unaddressed, may have an impact on the durability of
the wall.

Depending on the degree of pressure equalization, the distribution of
wind loads between the inner and outer wythe can vary considerably and
engineering judgement is required. A number of recommendations have
been provided in the literature:

From "Review of Design Guidelines for Pressure Equalized Rainscreen
Walls"[30]:
* The required area of weepers and vents to obtain around 75% pressure
  equalization is 1 – 2%.
* For a rainscreen wall system with an airtight air barrier system, the re-
  quired area of weepers and vents and well sealed small compartments
  (both horizontal and vertical), it is recommended that the rainscreen
  panel be designed for 70% of the maximum negative wind load and the
  air barrier for 90%. (Presumably these percentages apply to positive
  pressures as well.)

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\*     *The tie stiffness values from the Drysdale and Wilson report have been included in a data base that*
      *is part of the MVSS Finite Element computer program. Refer to the User Reference Manual.*

- For brick veneer wall systems with an airtight air barrier system, the required area of weepers and vents but with a continuous uncompartmented air space including the corners, it is reasonable to design (the rainscreen) for 90% of the negative wind loads near corners and 80% in the central half of a flat wall. (Presumably 90% for the air barrier still applies).

From "A Study of the Rainscreen Concept Applied to Cladding Systems on Wood Frame Walls"[31], pressure equalization can be studied using RAIN.EXE computer program. The program models dynamic wind loads and allows the user to study the effects of cavity volume, flexibility of the backup, vent area in the veneer and leakage area in the air barrier. Work is underway to correlate the output from this program with experimental results.

Little guidance is available regarding other difficult wind load issues:
- Should the full effect of wind gusts be applied to the air barrier?
- Do interior pressure coefficients apply to just the air barrier or should they be shared with the exterior wythe where pressure equalization is incomplete?

Engineering judgement is required.

### Strength of Air Barriers

The following references are suggested as suitable sources for determining resistances for various air barrier materials:

*a) Structural Requirements for Air Barriers*[32] includes a review of the following air barrier materials/systems:

- gypsum board
- rigid insulation
- polyethylene
- membranes

Note that this report contains a number of conservative assumptions with respect to loads and resistances for air barrier materials that may not be appropriate for the evaluation of existing buildings.

*b) Testing of Air Barrier Systems for Wood Framed Walls*[33] includes test results of a number of air barrier materials/systems for both strength and leakage. The test specimens relevant to MV/SS construction include:

- Fibreboard sheathing covered with Tyvek paper and strapping
- Glasclad insulation
- Exterior gypsum board and Perm-a-barrier tape
- Extruded polystyrene insulation
- Esclad Energy Envelope insulation

- Interior gypsum board with finished joints
- Sprayed in place polyurethane with fibreboard sheathing
- Polyethylene sheet sandwiched between fibreboard sheathing and interior gypsum board

### c) CAN/CSA – A82.20 Series–M91, Methods of Testing Gypsum and Gypsum Products[34] and CAN/CSA – A82.27 – M91 Gypsum Board[35]

For gypsum wall board, these standards provide resistances for nail pull through and for flexural failure:

Factored Resistance for drywall screw pull through = $\emptyset 28t$ N but not greater than 400 N for gypsum board up to 15.9 mm thick

Factored flexural resistance with paper fibres parallel to the span (i.e. board installed horizontally) $M_r = 12400\phi(t-1)$ N.mm/m.

Factored flexural resistance with paper fibres perpendicular to the span (i.e. board installed vertically) $M_r = 4150\phi t$ N.mm/m.

Notes:

i) $t$ = thickness in mm

ii) It is recommended[32] to use $\phi = 0.5$ particularly in view of some of the unconservative results reported in Note (iv).

iii) The flexural resistance of gypsum drywall is not necessarily lost with wetting and drying provided the paper has not delaminated from the gypsum core.

iv) The nail pull–through and flexural resistance expressions are minimum quality control requirements for the manufacture of drywall. Installed drywall should have higher strength but some lab tests (Testing of Air Barrier Systems for Wood Framed Walls[33]) indicate that these values are not always conservative. Removal of specimens from the field for testing may, therefore, be warranted. The CSA standards will provide some guidance with respect to test procedures although it is recommended that the flexural test procedure be modified to include a roller bearing at one end.

### 4.3.2.8 Interior Sheathing – Strength – Is It Good Enough?

Interior sheathing may behave as a deliberate or accidental air barrier. See Item 4.3.2.7, Air Barriers.

### 4.3.2.9 *Typical Stud – Strength – Is It Good Enough?*

The factored resistance for moment, shear and web crippling should be determined in accordance with the requirements of S136-94[36]. Refer also to CSSBI[37] for methods of analysis including a simplified method to account for warping torsional stresses between lines of bridging. Torsional stresses should be considered where the sheathings can not be relied on to resist the torsional component of load not applied through the shear centre.

Neither of the above documents addresses the weakening effect of cutouts for moment, shear and web crippling. Refer instead to the following references:

***a) Behaviour of Web Elements with Openings Subjected to Bending, Shear and the Combination of Bending and Shear[38]***

Typical industry web cut-outs create a relatively modest reduction in moment capacity usually in the order of 10% (compared with an unperforated web). Several analytical methods are proposed to predict this reduction. The simplest and most accurate approach treats the web above the cut-out as an unstiffened compression element.

The reduction in shear capacity due to the presence of web cutouts is quite severe and varies considerably depending on the geometry of the cut-out and the stud. Reductions in the order of 50% are common. Two predictor equations are proposed, a power equation and a bilinear approach. The bilinear equation is the simplest and most accurate.

An equation is also proposed for the unfavourable interaction that occurs between bending and shear.

***b) Structural Behaviour of Perforated Web Elements of Cold-Formed Steel Flexural Members Subjected to Web Crippling and a Combination of Web Crippling and Bending[39]***

Current industry practice is not to allow web perforations closer than 300 mm from the centre of the perforation to the end of the stud. This report confirms that this 300 mm dimension is approximately correct and also provides some web crippling reduction factors for situations where the cut-out is closer to the end of the stud.

*For end-one-flange loading conditions when the cut-out is not within the bearing length, the unperforated web crippling bearing strength shall be multiplied by the following reduction factor, RF:*

$$RF = 1.08 - 0.630(a/h) + 0.120(x/h) \leq 1.0$$

*where:*

$a$ = *cut-out depth (38 mm typically)*
$b$ = *cut-out length (102 mm typically)*
$h$ = *depth of flat portion of the web*
$x$ = *clear distance from edge of cut-out to edge of bearing*

An equation is also proposed for the unfavourable interaction that occurs between bending and web crippling.

MV/SS walls with studs that are not "good enough" can be reinforced as discussed in Chapter 5 or the structural capacity of the brick can be mobilized. Refer to MVSS finite element analysis, Item 4.2.4 or full scale testing Item 4.2.5.

### 4.3.2.10 *Typical Stud – Serviceability – Is It Good Enough?*

Refer to Items 3.1.1 – 3.1.4 for a discussion of the impact of flexible steel stud behaviour on brick cracking. In summary, the probability of brick cracking in a MV/SS wall system is high (but not certain). Flexural cracking may or may not result in an increase in water penetration through the veneer depending on the degree of pressure equalization. Except where the spacing between the first and second flexural crack is less than the spacing between ties ( a rare occurrence – see Item 3.1.2) flexural cracking be treated as a serviceability limit state, not a strength limit state.

For new construction, S304.1[27] requires that the veneer deflection be less than L/600 where L is the height of the steel stud backup. The veneer deflection is to include steel stud flexural deflections plus the mechanical play and deformation under load of the steel ties. The deformation of the top and bottom track details is not to be included in the L/600 check.*

For purposes of evaluating an existing MV/SS wall, the following accept/reject criteria are suggested:

- Apply all of the wind load to the steel stud backup and limit stud deflections to L/360. This deflection limit was accepted by a number of early industry standards and reports[40, 8, 41].

    Note that prior to the acceptance of L/360, a deflection limit of L/240 was occasionally specified. It was believed that L/240 represented, approximately, the deflection limit that the interior drywall could tolerate without suffering flexural cracking of its own.

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*   *A shortcut method is also allowed by S304.1[27] wherein the stud alone is checked for L/720 deflection. Limits are placed on the stiffness and mechanical play in the ties when this design approach is used.*

As discussed in Chapter 3, these earlier more relaxed deflection limits will likely result in increased water penetration through the veneer particularly where pressure equalization is poor.

- Analyze the wall system using the MVSS finite element computer program (See Item 4.2.4) and limit crack sizes to the current L/600 criterion. The relationship between crack size and stud deflection is illustrated in Figure 3 – 3 where:

$$\Delta_c = 4 \, \Delta_v \times (t/L)$$

Some relaxation of the L/600 limit would be justified in the absence of symptoms of distress due to water penetration through the exterior veneer. Engineering judgement is required.

- Conduct full scale in–situ tests of the wall and limit cracks as suggested for the MVSS finite element program or undertake water penetration tests in combination with the load tests. For details on water penetration testing, see Chapter 2.

Serviceability checks should be undertaken at specified wind loads (no load factor) using the lower NBC reference velocity pressure of $q_{(1/10)}$. Note that the CSSBI[37] recommends the use of $q_{(1/30)}$ when checking the steel stud backup for strength.

MV/SS walls with studs that are not "good enough" can be reinforced as discussed in Chapter 5 or the structural capacity of the brick can be mobilized. Refer to MVSS finite element analysis, Item 4.2.4 or full scale testing Item 4.2.5. Alternatively, the wall can be left as is and periodically monitored for symptoms of cracking distress.

### 4.3.2.11 *Bridging – Strength – Is It Good Enough?*

See Item 3.4 for discussion on bracing types and behaviour. See CSSBI[37] for methods of calculating loads and resistances.

Bridging that is not "good enough" can be supplemented with additional bridging or sheathing or replaced by retrofit ties that will restrain the studs from twisting. See Chapter 5. Alternatively, studs can be reinforced by installing new studs back–to–back with the existing studs. Back–to–back studs that meet S136–94[36] inter–connection requirements usually do not require bridging. See Chapter 5 (Item 5.2.13.1).

### 4.3.2.12 *Bottom Track – Strength and Serviceability – Is It Good Enough*

See Item 3.15 for a review of strength and serviceability issues.

The ability of bottom track to span as a flexural member between concrete fasteners should be checked using S136[36] for section properties and CSSBI[37] for analytical methods.

Bottom track must have enough resistance to local deformation such that it can mobilize the web crippling strength of the stud that frames into the track. Based on recent research at Cornell University[42], the resistance of the track to local deformation can be checked with the following formula:

$$P_{rt} = \phi \, 0.6 \, Fu \, w \, t$$

*where:*

| | |
|---|---|
| $P_{rt}$ | *= factored resistance of track to local failure* |
| *0.6 Fu* | *= ultimate shear strength of steel in track* |
| *w* | *= flat width of stud flange* |
| *t* | *= thickness of track* |
| $\phi$ | *= 0.75 (say)* |

*Notes:*

*i) For studs back to back or toe to toe use w = flat width of one stud*
*ii) For studs near the end of track use 70% of $P_{rt}$*
*iii) These are interim design values. Final report has not been issued.*
*iv) Tests were based on a 6 mm end gap from end of stud to track*

See Drysdale and Breton[12] for additional information on the strength of track for end gaps up to 12 mm.

For serviceability performance of track see Pekoz[42] and Drysdale and Breton[12]. Generally the flexibility of the stud to track connection increases with increasing end gap. Note that the flexibility of the bottom track has little effect on the performance of the veneer to the extent that the first cracking load for the veneer is relatively insensitive to changes in bottom track stiffness[12]. As a general rule, if the strength of the stud to track connection is adequate, then the serviceability of the connection can also be deemed to be "good enough".

Where the bottom track is not "good enough" studs can be connected directly to the concrete. See Chapter 5.

### 4.3.2.13 *Top Track Assembly – Strength and Serviceability – Is It Good Enough?*

See Item 3.16.

Analytical methods for calculating the resistance of the top track assembly to load are described in CSSBI[37]. The CSSBI document recommends

checking the leg of the outer top track using the somewhat conservative elastic section modulus $S = (1/6)bt^2$. For the evaluation of existing buildings the plastic section modulus $Z = (1/4)bt^2$ might be more appropriate but only if the non-uniformity of load is checked using the beam on an elastic foundation analogy in CSSBI Appendix E.

Top track deflections under wind load are of little consequence in the performance of the veneer but may affect the performance of caulking or air barrier materials.

If the inner and outer top track detail is not "Good enough" it can be difficult to reinforce. See Chapter 5.

### 4.3.2.14 *Built-up Jamb Studs - Strength and Serviceability - Is It Good Enough?*

See Item 3.17.

To check the strength and flexibility of these members see CSSBI[37], S136[36] and the discussion of web cutouts in Item 4.3.2.9.

Jamb stud problems can be frequently overcome by installing retrofit ties to the jamb and spreading the load to neighbouring studs. Where jamb studs are not interconnected, retrofit ties to both members of the built-up section can insure they work together. This repair solution is explored in the separate MVSS User Reference Manual[9], Case Studies 2 – A and 2 – B. See also Chapter 5.

### 4.3.2.15 *Head and Sill Members - Strength and Serviceability - Is It Good Enough?*

See Item 3.18, CSSBI[37] and S136[36].

Head and sill member problems can be frequently overcome by installing retrofit ties and/or by mobilizing the horizontal bending strength of the veneer.

### 4.3.2.16 *Parapets - Strength and Serviceability - Is It Good Enough?*

See Item 3.19 and CSSBI[37].

For parapet veneer problems the repairs are similar to those required for the typical floor to floor veneer. For steel stud backup problems, repair usually means replacement.

Note that the disruption to the water proofing membrane and flashings on the roof can make parapet repairs/replacement difficult and expensive.

### 4.3.2.17 *Sealants – Serviceability – Is It Good Enough?*

See Item 3.20.

Sealants can be deemed to have failed and require replacement if they no longer act as a rain barrier or if they can no longer accommodate the movement at the caulked joint.

### 4.3.2.18 *Corrosion – Strength – Is It Good Enough?*

See Item 3.21.

As discussed in Item 3.21, corrosion can be categorized as one of the following:
- White rust – the result of oxidation of free zinc on the surface of the member and usually means some zinc has been lost but some remains.
- Brown rust – the result when all of the protective zinc has been lost and the parent material is starting to corrode. Brown rust can vary from a few isolate spots to a substantial portion of the area of the part.
- Brown rust with loss of parent material – the most severe condition and is usually a concern structurally.

A corroded steel part can be deemed not "good enough" if:

a) The loss of parent material has significantly compromised the strength of the member.
b) The loss of galvanizing has significantly compromised the expected future service life for the wall. Refer to S478[20] for guidance on establishing service life and corrosion performance.

Where corrosion has significantly compromised strength of the steel stud backup, reinforcement can be added or the plate action of the brick can be mobilized to reduce the load on the affected part. Corroded ties can be corrected by adding retrofit ties. See Chapter 5.

Where corrosion has compromised the future service life the rate of deterioration can be reduced to achieve the expected service life. This approach usually requires a significant building science performance upgrade. Zinc rich paint can also extend the service life of some corroded components. See Chapter 5.

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# *Chapter 5 – Maintenance and Restoration Strategies*

In order to successfully maintain or restore a problem wall, the investigator must address both financial and technical issues.

In this chapter, Section 5.1 discusses the maintain, restore or replace options in an overall context of risk and financial management. The investigator is provided with some guidance on the optimum course of action that best meets the needs of public safety, building performance and the financial position of the owner.

The technical aspects of a number of possible restoration strategies are addressed in Section 5.2. This section should be read in conjunction with the information presented under similar category headings in Chapters 3 and 4. Cross referencing is provided, where appropriate.

## 5.1  Economic Analyses*

### 5.1.1  Background

Decisions on the types of repairs or maintenance to be completed on a particular building, and the timing of the work are rarely made solely on the basis of a technical assessment. Costs associated with the repair are the major consideration of the building owner when deciding on how to proceed. Conversely, financial planning for the repair can not occur without careful consideration of the physical conditions and technical assessment of the problems and expected performance.

If accurate decisions are to be made that suit the business and financial position of the owner, the professional technical specialist must understand and contribute to the financial analysis that will ultimately identify how the wall system will be managed.

### 5.1.2  Cash Flow Options

Building owners normally desire options for implementation of  repairs that result in options for the required cash flow.  This is most common with large projects where the owner does not have immediate access to the required capital.

Deferring the work for a period of time to allow finances to be arranged is almost always necessary to some degree.  Depending on the length of deferral, difficulties arise in trying to predict  the degree of ongoing damage or safety issues that may develop.

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Phasing the required repair over a number of years also provides some leeway in financing the work. Similar to deferral, the technical concerns related to areas of walls not repaired for a period of time must be considered. If variations in the severity of problems have been identified over the surface of the building, it may be possible to tailor a repair to defer work at specific areas of the building for as long as possible. In the extreme, this becomes a repair on an "as-needed basis". For example, the repair procedure might be to only repair leakage problems one at a time as each becomes apparent.

## 5.1.3  Costs To Be Considered

Costs to be considered when analysing repair options should not stop at the estimates of capital required to implement the work. Other costs related to ongoing ownership of the building should be identified to allow a fair comparison of all options.

### 5.1.3.1  *Costs Related to Deferring the Work*

Various budgets may need to be made available to allow the repair to be deferred. These could include costs related to providing protective hoarding to limit access to dangerous areas by the public, ongoing inspection to monitor continuing deterioration and to check for dangerous conditions developing, local repairs to address immediate hazards that develop or severe leaks that must be mitigated, or losses related to ongoing leakage. These types of costs can easily become prohibitive where general defects exist.

Other costs that should be considered relate to a loss of efficiency in completing the work. Phasing the work over a period of time may escalate costs related to the contractor executing the work, and for administration and inspection by the consultant. Managing generally deficient conditions on an as-needed basis can often result in substantial inefficiencies and high costs.

Inflation of costs should also be considered. Low construction costs during a recessionary time can be advantageous. There seems to be a trend for a general escalation of construction costs as a result of increasing awareness and restrictions related to health and safety and the environment. For example, costs of disposal of materials is rapidly escalating in many areas.

### 5.1.3.2  *Initial Capital Cost of the Repair*

The initial capital cost of the repair must be determined with some degree of certainty. Obtaining bids from experienced contractors with a defined scope of work and specification is the best method of ensuring that

accurate costs are used in the analysis.

In some circumstances, obtaining more than one bid may be desirable. For example, financial analysis based on initial estimates may indicate only small differences between a number of options under consideration. Obtaining bids for the various options and reworking the financial analysis will improve the chances of making the right choice.

### 5.1.3.3 *Energy Conservation*

Repair options may present an opportunity to improve the performance of the building and return at least part of the capital to the owner in the form of energy savings. Upgrading of insulation and reducing air leakage can directly save on heating and cooling. A change in the exterior surface during re-cladding may present an opportunity for reducing solar gain. Incorporating solar collectors into the cladding system can contribute to building heating.

### 5.1.3.4 *Maintenance Repairs*

Maintenance repairs of a wall system can represent a significant cost. Costs may be offset if the repair results in a renewal of components and deferral of future maintenance. For example, new cladding on top of the existing veneer should avoid the need for future re-pointing of mortar joints or renewal of the caulking.

## 5.1.4 Costs Versus Risks

Once costs for various repair options are well defined, these become a very persuasive factor in comparing one repair to another. It is important that the risks associated with each repair are equally well defined. Low cost repairs (such as local repairs or use of sealers) typically have more associated risks as compared with high cost repairs (such as re-construction or re-cladding). These risks should be identified, and estimates of further work that might be required considered. The possibility of ongoing further costs may clearly identify an initial low cost option as not being desirable. Conversely, an initial low cost option with even a limited potential for success and few risks may be an obvious candidate for at least a trial repair program.

Examples of risks are:

* *Hazardous Conditions*

   The liability associated with knowing that there is a possibility of hazardous conditions existing or developing that could endanger public safety can often eliminate a particular repair option. Alternatively, risk management may be possible to allow continued consideration of the option. Risk management may

involve inspection or monitoring or temporary safety measures to address the identified concerns.

- *Incomplete Treatment*

  Areas outside of the areas initially identified as requiring treatment may subsequently show symptoms of distress and require treatment. Costs result from the necessity to re-assess, investigate, access and repair the new areas.

- *Ongoing Deterioration*

  If all of the conditions that contributed to the problems are not properly addressed, ongoing problems may occur in areas not treated, or in repair areas.

- *Un-Manageable Repairs*

  Some repairs may result in un-manageable wall systems owing to the complexity of conditions that are created. Progressive patching or use of different repairs over time leads to varied wall conditions. Ongoing monitoring, inspection, identification of problems, and repair can become complex and impractical. A more general program of repair may become necessary to provide consistent and acceptable performance. For example, programs of spot repair of caulking are often embarked upon, leading to varied conditions as a result of differences in sealant material, material weathering and workmanship. Ongoing leakage as original materials fail and problems develop at the various joints between old and new caulking often leads to a need for general re-caulking program.

- *Failed Repair*

  There is often at least some risk that widespread problems following a repair will continue to exist or develop. This risk can normally be reduced by further investment in investigation and testing, or by implementing trial repair programs and monitoring to prove success.

  A failed repair does not necessarily mean that money was unwisely spent. If the work completed was necessary for other reasons such as normal maintenance, and if the subsequent repairs found to be necessary to address the problem do not negate the value of this work, there is little risk. For example, re-caulking a building with leakage problems that is developing general caulking defects would be a reasonable first repair attempt even if subsequent repair of internal flashing defects were to be identified as being necessary.

### 5.1.5  Methods of Economic Analysis

In order to fairly compare future and present costs, some method of considering the effects of inflation and interest earned on capital is necessary.

### 5.1.5.1 *Present Value*

Costs are generally related to the value of money today. A future cost generally has less value than an equal amount spent today as a result of the effects of inflation and interest. A present value analysis calculates the values of costs and savings in current dollars so that they may be summed to allow comparison between options.

The effects of interest rates and inflation can be considered using the following calculations:

To estimate the future cost of a repair assuming a constant annual inflation of the costs, where "inf" is inflation and "$n$" the number of years:

$$\text{Future Value} = \text{Present Value} \times (1 + \text{inf})^n$$

To calculate the present value of a future cost or savings assuming a constant rate of return on investment, where "$i$" is the interest rate and "$n$" the number of years:

$$\text{Present Value} = \text{Future Value} \times \frac{1}{(1 + i)^n}$$

To calculate the present value of a fixed annual cost or savings assuming a constant rate of return, where "$i$" is the interest rate and "$n$" the number of years:

$$\text{Present Value} = \text{Annual Amount} \times \frac{(1 + i)^n - 1}{i(1 + i)^n}$$

Present value analysis allows and "apples to apples" cost comparison between a number of alternatives using today's dollars. Further analysis may be required to consider the additional costs associated with financing the work.

### 5.1.5.2 *Reserve Fund Planning*

Reserve fund planning is common in the condominium industry. The concept is to calculate all expected major repair and replacement costs necessary to maintain the building. These costs are then used to calculate the annual contribution amount to a reserve fund that is necessary to provide adequate funds to finance the work. Good reserve fund planning results in a consistent level of annual contribution, avoiding the need for any abrupt change in the fees charged to owners.

Reserve fund planning can be used to compare the costs of repair options with the best option typically defined as the one with the lowest annual contribution amount. For example, large capital projects that eliminate the need for or defer future work may reduce the required annual contribution. Alternatively, low cost projects that defer major capital expenditures beyond an initial period of limited funds being available can also dramatically reduce the annual contribution amount required. Note that the best repair option as identified by this process may be different from the least cost option from present value analysis.

Calculation methods can vary from a simple but conservative "linear analysis" to a more accurate "cash flow" analysis.

The linear analysis method is most commonly employed because of its simplicity and compatibility with spread sheet type software programs. This approach sums the annual amounts necessary to accumulate the budget for each individual project when it is required. For example, a $100,000 repair item that occurs in 10 years would require $10,000 per year in annual contributions. If $50,000 had already been accumulated towards the repair, $5,000 per year would be necessary to accumulate the required fund. The example neglects interest and inflation. If included, these factors would result in a slightly lower annual contribution amount.

The "cash flow analysis" method requires special computer software. Rather than summing the individual contributions necessary for each individual repair item, an analysis of the cash flow for all items is carried out to determine the minimum annual contribution necessary to avoid a deficit. This will result in certain "critical years" being identified where the reserve fund is depleted by major capital investments. This analysis method easily lends itself to considering the effects of interest earned on the reserve fund, or gradually increasing the annual contribution amounts over time.

In some cases, the cash flow analysis method will calculate an annual contribution that is similar to a linear analysis. Results are similar when calculating the annual contribution amount for a new building or when calculating the annual contribution amount for an older building that has been adequately funded in the past. However, in many cases the cash flow analysis can result in lower annual contribution amounts. This is particularly true where the reserve fund has been underfunded for a period of time, or where unexpected repairs must suddenly be worked into the plan.

As a simple example of how the cash flow analysis method can calculate a lower annual contribution amount, consider a building with only two repair activities to be budgeted for and no existing reserve fund. Each is expected to cost $100,000, the first repair is anticipated to be necessary in 2 years, the second is anticipated in 4 years. Neglecting the effects of

interest and inflation, a linear analysis would calculate that the necessary annual contribution amount is $75,000 per year ($50,000 towards the first repair, and $25,000 per year towards the second repair). A cash flow analysis would calculate that sufficient funds for the repairs could also be provided by an annual reserve fund contribution of $50,000. The cash flow analysis therefore allows establishing a tighter financial plan.

For more detail on the linear versus the cash flow analysis, refer to Loeb[1].

## 5.2  Compendium of Restoration Procedures

This section presents a number of possible maintain or restore strategies for problem walls.

Information can be located in this section by using the index in Table 5 – 2 which directs the reader to the appropriate item discussed in the text. The table lists alphabetically:
- Symptoms observed
- Wall element under distress, or
- Restoration procedure

| TABLE 5 - 2  Index - Refer to Item Numbers Indicated | |
|---|---|
| **Wall Component or Symptom** | **Strength, Seviceability or Building Science** |
| Air barrier | *See 5.2.9, 5.2.10, 5.2.13.2* |
| Air leakage | *See air barrier* |
| Air space | *See 5.2.8* |
| Brick | *See 5.2.1.2, 5.2.5, 5.2.13.2 (Figure 5 - 16)* |
| Bridging | *See 5.2.6, 5.2.13.1, 5.2.14* |
| Caulking | *See sealants* |
| Cavity | *See air space* |
| Cleaning | *See 5.2.5.10* |
| Colour treatment | *See 5.2.5.9* |
| Control joints | *See cracking* |
| Corrosion | *See 5.2.20* |
| Cracking – flexural | *See 5.2.5.1* |
| Cracking – horizontal movement | *See 5.2.4* |

| Wall Component or Symptom | Strength, Seviceability or Building Science |
|---|---|
| **TABLE 5 - 2** <br> *Index - Refer to Item Numbers Indicated* | |
| Cracking – vertical movement | *See 5.2.1.3, 5.2.3* |
| Deflection | *See studs - typical and built-up, head track, sill track, strong post, cracking - flexural,* |
| Drywall – exterior | *See sheathing - exterior* |
| Drywall – interior | *See sheathing - interior* |
| Drying | *See 5.2.5.1, 5.2.5.7, 5.2.6* |
| Dust shadowing | *See insulation - exterior* |
| Efflorescence | *See 5.2.5.5, 5.2.5.9, 5.2.5.10* |
| Expansion joints | *See cracking* |
| Flashings and Damproofing | *See 5.2.5.8, 5.2.7* |
| Freezing and Thawing | *See 5.2.5.6, 5.2.5.9* |
| Graffiti | *See 5.2.5.10* |
| Head track | *See window* |
| Insulation – batt | *See 5.2.11* |
| Insulation – exterior | *See 5.2.9, 5.2.10* |
| Lintels | *See 5.2.2* |
| Moisture | *See 5.2.5.1, 5.2.5.3, 5.2.5.5, 5.2.5.6, 5.2.5.7, 5.2.5.8, 5.2.5.9, 5.2.5.10, 5.2.6, 5.2.8, 5.2.9* |
| Moisture movements | *See movement* |
| Mortar and mortar joints | *See 5.2.5.3, 5.2.5.4, 5.2.8* |
| Movement | *See cracking, top track* |
| Parapets | *See 5.2.19* |
| Rain screen | *See 5.2.5.1, 5.2.5.7* |
| Sealants | *See 5.2.4 (Figure 5 - 3), (Figire 5 - 9)* |
| Sealers and Surface Coatings | *See 5.2.5.9, 5.2.8* |
| Sheathing – exterior | *See 5.2.9, 5.2.10* |
| Sheathing – interior | *See 5.2.10, 5.2.12* |
| Shelf angles | *See 5.2.1, 5.2.5.2, (Figure 5 - 21)* |

| Wall Component or Symptom | Strength, Seviceability or Building Science |
|---|---|
| Sill track | *See window* |
| Spalling | *See freezing and thawing* |
| Strong Post | *See 5.2.13.2* |
| Studs – built–up at jamb | *See 5.2.17* |
| Studs – typical | *See 5.2.13, 5.2.15* |
| Thermal bridging | *See 5.2.6, insulation - exterior, insulation - batt* |
| Thermal expansion and contraction | *See movement* |
| Thermal – R value | *See insulation - batt, insulation - exterior* |
| Ties | *See 5.2.6, 5.2.13.2, 5.2.14* |
| Track – bottom | *See 5.2.13.3, 5.2.13.4, 5.2.15* |
| Track – top assembly | *See 5.2.13.4, 5.2.15, 5.2.16* |
| Vapour retarder | *See air barrier* |
| Vents and weepers | *See 5.2.5.7, 5.2.5.8* |
| Weepers | *see vents* |
| Window – head and sill members | *See 5.2.18* |

**TABLE 5 - 2**
*Index - Refer to Item Numbers Indicated*

### 5.2.1  Shelf Angles

See also,  Items 3.2 and 4.3.2.1.

#### 5.2.1.1  *Replacement*

When all of the brick is removed from the building, replacing a shelf angle is similar to installing a shelf angle on a new building. Drilled–in concrete anchors may be required if cast in adjustable anchors are deemed inadequate.

When the brick veneer is to be retained, it is necessary to create openings in the veneer and remove and replace the shelf angle in stages. See Figure 5 – 1. Cut openings "a" and remove and replace the shelf angles

at these locations. Replace the brick and proceed in a similar fashion with locations "b".

Existing brick veneer

Existing shelf angle

A          B          A
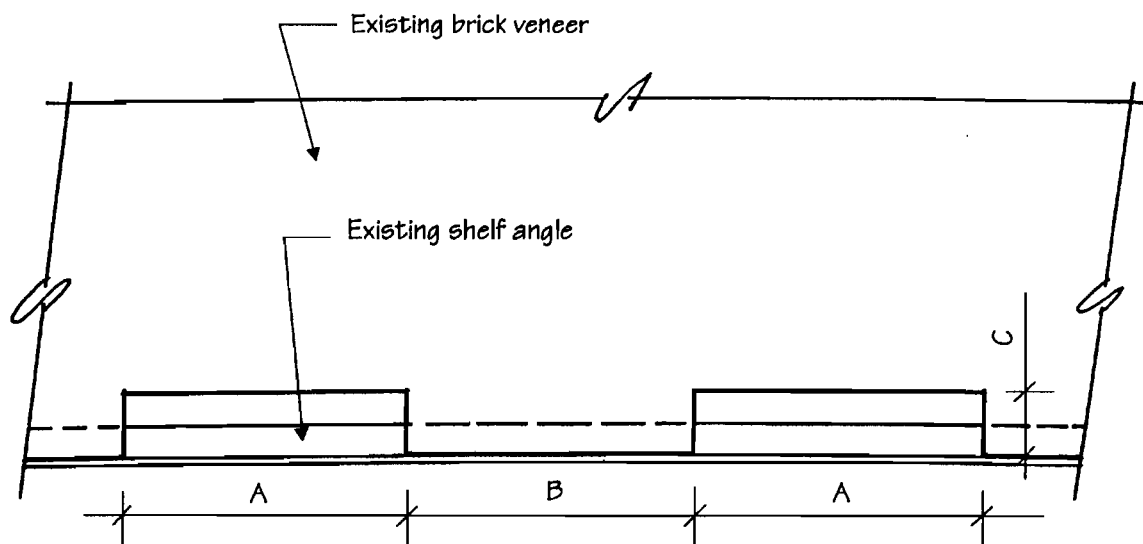
NOTES:

1. Dimension "A" = width of opening in veneer not to exceed the arching capability of the brick (1m typically)

2. Dimension "B" = "A" typically.

3. Dimension "C" = height of opening in veneer (3 courses of brick typically)

FIGURE 5 - 1     SHELF ANGLE REPLACEMENT WITH VENEER
                 IN PLACE

Rather than leaving existing brick at locations "b", some contractors prefer to use jacks or timber supports. The procedure is the same, however, with removal and replacement at "a" followed by "b".

The original flashing will also require replacement – see Item 5.2.7.

Since the existing angle has been deemed inadequate, it is likely that the soft joints (if any) beneath the angles are inoperative and compressive stresses have developed in the veneer. Repairs must be undertaken from the top down, relieving stresses on the lower floors as the repair proceeds.

Note that the absence of a soft joint can be a virtue when undertaking this type of repair since the veneer below helps to support the defective shelf angle and the weight of the veneer above. If a soft joint exists, it may be prudent to temporarily shim to the underside of the shelf angle to mobilize this support below.

### 5.2.1.2  *Leg Extension*

Where the overhang of brick is deemed to be excessive, the horizontal leg of the angle can be extended. See Figure 5 – 2a and 5 – 2b.

Shelf angle

Inermittent fillet weld

Continuous plate chamfered to accommodate radius at heel of angle

Extension

5 - 2a

Shelf angle

Intermittent full penetration groove weld

Extension

5 - 2b
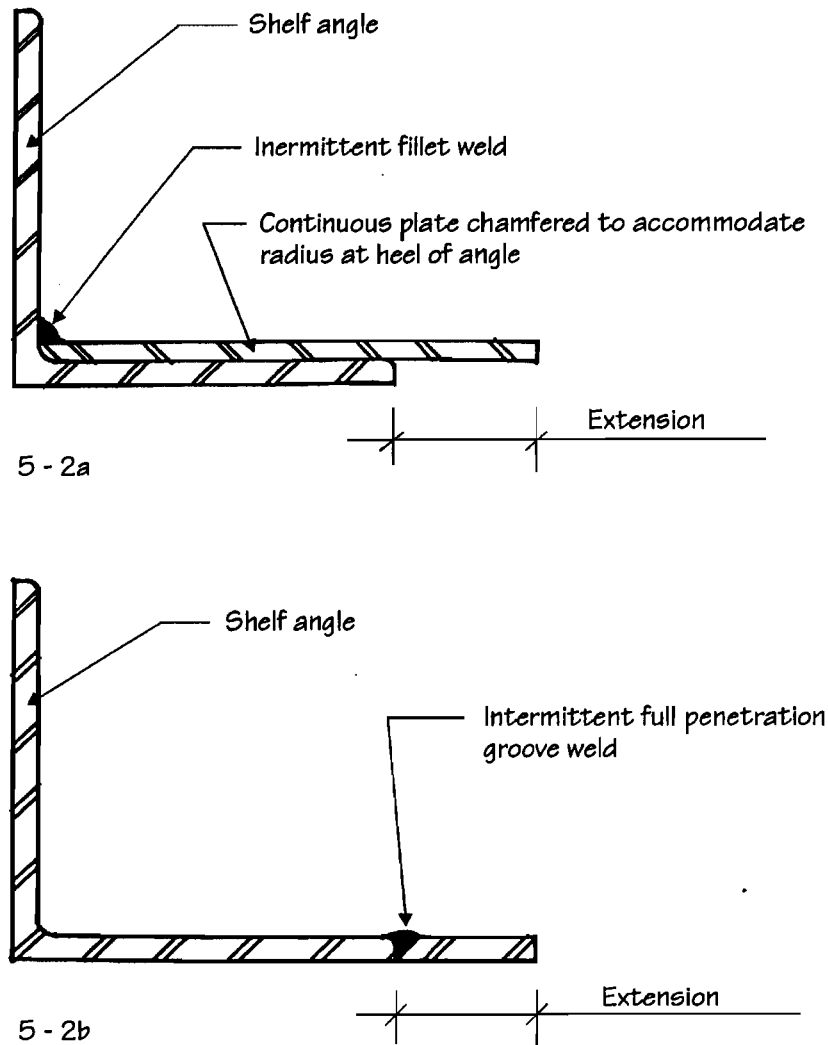
NOTES:

1. Where welding from above, provide staggered openings in the veneer similar to Figure 5 - 1.
2. Insure that existing shelf angle and shelf angle to concrete connectors have adequate capacity.
3. The extension in Figure 5 - 2b could be welded from above (as shown) or below. Working from below requires a difficult overhead welding procedure but simplifies brick removal. Typically, only the top one or two courses beneath the shelf angle need be removed.

FIGURE 5 - 2        SHELF ANGLE HORIZONTAL LEG EXTENSION

### 5.2.1.3 *Discontinuities*

Where the discontinuities are small, shelf angle extensions can be welded on, perhaps with no additional concrete fasteners. Large discontinuities will require a new angle with concrete fasteners.

Vertical corner cracking due to discontinuous shelf angles (see Figure3 – 13) usually requires the removal and rebuilding of the corner sections including continuous shelf angles around the corner. Where the shelf angle discontinuity is small, it may be possible to ignore the discontinuity in the shelf angle and continue the horizontal movement joint around the corner to relieve the stresses in the veneer.

### 5.2.1.4 *Corrosion*

Shelf angle corrosion can frequently be arrested by installing new flashing (see Item 5.2.7). Some investigators believe that peel and stick type membranes work well as flashings and provide good corrosion protection to the shelf angle. Treatment with zinc–rich paint may also be appropriate. Severely corroded angles may require replacement. See also Item 5.2.20.

### 5.2.1.5. *Concrete Connectors*

Inadequate concrete connectors (inadequate strength, poor shimming etc.) can be upgraded by ignoring the existing connectors and installing new drilled in anchors with the necessary shims. Small openings in the brick will be required at each fastener location.

## 5.2.2 Loose Angle Lintels

See also Items 3.3 and 4.3.2.2.

### 5.2.2.1 *Replacement*

Loose angle lintel replacement typically requires the removal of the brick above. In most MV/SS, the brick above (for windows) is only a few courses

### 5.2.2.2 *Leg Extension*

See Item 5.2.1.2

### 5.2.2.3 *Corrosion*

See Item 5.2.1.4 and 5.2.20.

### 5.2.3 Brick Veneer Vertical Movement Joints

See also Items 3.4 and 4.3.2.3.

Vertical movement joints can be saw cut into existing veneer.

- The new joints should be located at or near the locations where cracking has occurred. See Drysdale and Suter[2] for additional guidance on locating joints.
- The new joints are cut with a masonry saw in one or two passes depending on the blade thickness and the required width of joint. Typical retrofit joint widths range from 10 to 15 mm.
- The veneer requires support on either side of the saw cut joint. A saw cut joint between stud locations is satisfactory provided the adjacent studs and brick ties have adequate capacity. See Item 5.2.6 where retrofit ties to steel studs are required. Alternatively, the saw cut can be installed at the location of a concrete column or shearwall with retrofit ties installed to the concrete (if required).
- Note that introducing a vertical control joint near a corner may result in increased tie loads since the brick spans horizontally to the tie adjacent to the saw cut rather than to the corner. Introducing a control joint in the plane of the wall, away from corners, is not likely to result in higher tie loads adjacent to the new control joint.

As a short term solution, the cracks that have been formed can be caulked and treated as the new movement joint. This approach is not satisfactory in the long term because:
- Debris in the crack may compromise its ability to function as a movement joint.
- Cracks are often an inappropriate dimension for caulking
- The crack may close completely after caulking and force out the sealant material.
- The repair is usually unsightly.

### 5.2.4 Brick Veneer Horizontal Movement Joints

For behaviour, deficiencies and consequences see Item 3.4.

Retrofit soft joints (10 – 12 mm typically) can be installed beneath shelf angles but only when the shelf angle and its anchorage are structurally adequate and the brick overhang is not excessive. Repairs must be undertaken from the top down, relieving stresses on the lower floors as the repair proceeds.

The soft joints are typically created by removing one brick course beneath the shelf angle, trimming the bricks to the appropriate dimension and replacing. Where the original bricks cannot be salvaged or where it is not economical to do so, then new matching bricks cut to the required dimension will be necessary. Note that with the large assortment of brick sizes currently available, it may be possible to find a new brick that can be installed without any cutting required.

Horizontal movement joints can also be saw cut without removing any bricks. This approach can be economical but has a number of potential disadvantages:

• The release of locked-in compressive stresses can result in frequent saw binding.

• The saw cut joint can be difficult to inspect. If the saw joint is not cut completely through the veneer, an eccentric load path will result. This eccentricity could precipitate buckling of the veneer where none occurred before.

After installation of the horizontal movement joint, caulking is required. See Figure 5 – 3.
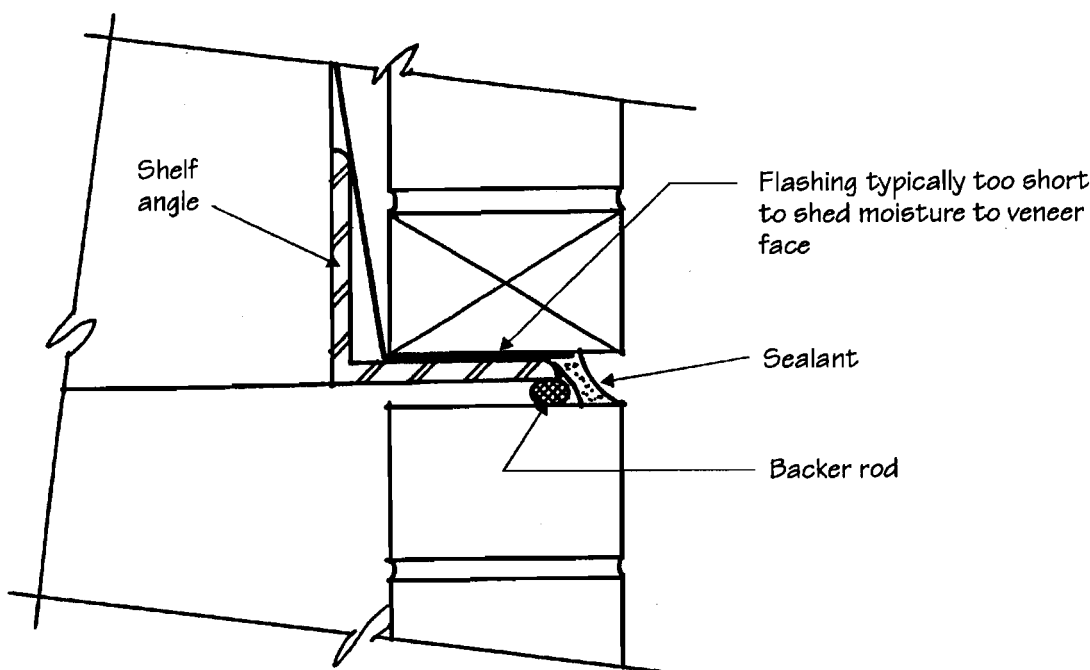


NOTES:

1. Install sealant to:

  a) prevent water from running back under shelf angle
  b) prevent water from entering top of brick below
  c) maintain approximate 2 : 1 height to thickness ratio for sealant profile.

FIGURE 5 - 3      SUGGESTED RETROFIT SEALANT DETAIL

It is prudent to delay the installation of the caulking for a period of time. This delay allows an inspection of the new horizontal movement joint to see if it has closed at

any locations. Where it has closed, it is likely that the shelf angle or its connection is inadequate and additional remedial work will be required.

### Are New Soft Joints Required at Every Level?

Occasionally, soft joints are installed at every second or third floor level rather than at every level. This approach, while economical, has a number of disadvantages:

* The loads on the shelf angle at each movement joint are difficult to determine analytically. These loads will include some portion of the entire weight of the veneer between movement joints aggravated by compressive stresses that remain locked in to the veneer.
* The compressive stresses remaining in the veneer may still be a problem with respect to local spalling at the intermediate shelf angles or overall buckling of the veneer. Note that these remaining compressive stresses and the load on the shelf angle could be estimated using the Fenton and Suter computer model discussed in Item 4.3.2.3 (b).

### 5.2.5  Brick and Mortar

#### 5.2.5.1  *Flexural Cracking*

See also Items 3.5.1 and 4.3.2.4(a).

If flexural cracking is a concern, it is usually with respect to water penetration through the veneer. However, it is unlikely that symptoms of water penetration would be due to flexural cracking alone.

Where water penetration through flexural cracking is a concern, it can be minimized with the following strategies:

* For simple veneer panels spanning from slab to slab, the cracking usually occurs at a horizontal mortar joint near midheight. The mortar joint could be raked back and caulked. Note, however, that the flexural cracking is typically hairline and difficult to identify.

    Where boundary conditions and/or openings complicate the flexural behaviour of the brick, the pattern of horizontal cracking might be too complex for this approach.

* Pressure equalization could be improved. This can be achieved by reducing air leakage through the air barrier, adding or enlarging vents and weepers in the veneer or compartmentalizing the air space with vertical baffles.

* Wall drying could be improved by adding or enlarging vents and weepers.

- The ability of the wall to handle the extra water penetration (if any) can be upgraded with improved flashings (see Item 5.2.7) and weepers (see Item 5.2.5.7).

### 5.2.5.2 Brick Overhang

See also Items 3.5.2.1 and 4.3.2.4(b).

See Item 5.2.1.2.

### 5.2.5.3 Brick Cracking – Miscellaneous

See also Item 3.5.3.1.

The causes of brick cracking are discussed in Item 3.5.3.1 and include to poor bond between the mortar and brick units, flexural cracking, horizontal movement or vertical movement. For horizontal and vertical movement, see Items 5.2.3 and 5.2.4. For flexural cracking, see Item 5.2.5.1.

Cracking due to poor mortar/brick bond is a potential path for moisture to enter the cavity and can be dealt with as discussed in Item 5.2.5.1 (flexural cracking). This type of cracking may be due to improper construction techniques or choice of materials (see Item 3.5.3.1) and may occur at several joints simultaneously. There may be a risk of veneer instability between ties (see Figure 3 – 2). Retrofit ties could be added to stabilize the veneer.

### 5.2.5.4 Mortar Joints

See also Items 3.5.3.2 and 4.3.2.4(c).

Deteriorated mortar should be cut out and repointed. Remove old mortar to a depth of 20 mm ± or until sound mortar is reached. Ideally, the new mortar should match the type, strength, colour and texture of the existing mortar although this can be difficult to achieve in practice. Colour matching may be facilitated if the veneer is cleaned first (see Item 5.2.5.10). Latex modified mortars are sometimes preferred for re-pointing for improved resistance to water penetration and better adhesion to the brick and existing mortar.

As is the case for new masonry construction, a well tooled concave, V-joint or weathered joint is preferred. See Drysdale and Suter[2] for various joint types.

### 5.2.5.5  *Efflorescence*

See also Item 3.5.4.1

Efflorescence is best treated by reducing the amount of moisture that is penetrating the brick. This moisture can have many sources. See Item 3.5.4.1.

Moisture penetration into the brick as a result of wind–driven rain may be a contributor to efflorescence and in some cases, sealers applied to the brick may improve performance. (See Item 5.2.5.9, Sealers.) However, this approach can also have serious negative consequences. Sealers may result in the crystallization process occurring in the pore structure beneath the surface of the brick instead of at the surface. The stresses generated from crystallization work repeatedly against the walls of the pores and are sufficient to fracture the brick resulting in face spalling. This mechanism is similar to freeze thaw spalling except that in the case of salts the pressures are much larger[3]. Note that spalling once started tends to occur at an accelerating rate since bricks that are already spalled are less resistant to moisture penetration.

Some investigators have argued that where moisture penetration due to wind–driven rain is the culprit, repointing is preferred to sealers (see Items 5.2.5.4 and 5.2.5.9). In either case, it is prudent to do a field trial of the selected approach on a portion of the wall.

For cleaning efflorescence, see Item 5.2.5.10, Cleaning.

### 5.2.5.6  *Freeze Thaw Damage*

See also Item 3.5.4.2.

For behaviour, deficiencies and consequences see Item 3.5.4.2.

Freeze thaw damage is the result of brick with inadequate freeze–thaw resistance combined with saturation near the limit of the brick adsorption capacity. Freeze thaw is best treated by reducing the amount of moisture that is penetrating the brick. This moisture can have many sources – see Item 3.5.4.1. Spalling once started tends to occur at an accelerating rate since bricks that are already spalled are less resistant to moisture pene-tration from wind–driven rain.

Sealers can be both beneficial and detrimental (see Item 5.2.5.9). They help keep out moisture from wind–driven rain but have a limited breathing ability and can therefore only vent a limited amount of moisture from within the brickwork. Typically, coatings cannot cope with a significant amount of moisture and the freeze thaw problem is likely to persist or be

worse where moisture concentrations are the greatest[4]. A field trial on critical portions of the wall is prudent.

A new insulated exterior cladding can be added to keep the brick above freezing temperatures and to eliminate exposure to wind-driven rain.

### 5.2.5.7   *Vents and Weepers*

See also Item 3.5.4.3.

For behaviour, deficiencies and consequences see Item 3.5.4.3.

Weepers are frequently plugged by mortar droppings in the air space. Weepers can be re-established by:
- drilling out a path through the mortar droppings if the droppings are not too deep
- establishing a new weeper at the next brick course up by cleaning out a head joint and drilling if required
- removing the brick adjacent to the weeper, cleaning out the droppings and re-establishing the brick.

Vents can be added to improve pressure equalization and wall drying. Size and spacing typically match the size and spacing of weepers. Vents should include a baffled screen to inhibit the entry of wind-driven rain and insects.

### 5.2.5.8   *Damp-proofing*

Symptoms of inadequate damp-proofing typically occur in the first few course of brick at the foundation level (efflorescence and/or freeze thaw spalling). Remedial action might include:
- Installation of through the wall flashing on top of the foundation wall. See Item 5.2.7, Flashing.
- Lowering exterior grade for 150 - 200 mm clearance between the brick and grade level
- Sloping exterior grade away from the building
- If there is a basement, waterproofing the foundation wall for say 600 mm below grade.

### 5.2.5.9   *Sealers and Surface Coatings*

Sealers have the potential to offer a very economical fix to a problem wall. See Item 5.2.5.5, Efflorescence, Item 5.2.5.6, Freeze Thaw and Item 5.2.8, Air Space for some examples where sealers might be appropriate.

There are a wide variety of possible coatings available.
- Cementitious Coatings such as stucco
- Paints including cement-based, latex and alkyd types

• Clear surface treatment repellents including silicones and acrylics.

### Clear Surface Treatment Repellents

Clear surface treatment repellents are the most common application and can be classified a number of ways:
• Water or organic solvent based
• Surface or penetrating
• Breathable or not
• Alkali resistant or not

The chemistry of clear surface treatment repellents is typically quite complex and a successful application may rely on some additional chemical reaction with the masonry itself. For example, some products rely on a reaction with moisture in the wall in the presence of an appropriate level of alkalinity. These conditions do not always exist or they may exist in only part of the wall. The result may be an effective seal for the alkaline mortar but no seal for the neutral brick. Consultation with the manufacturer and expert application are essential.

The silicone water repellents can be further subdivided into silicone resins, silanes and siloxanes. These treatments change the contact angle between the water and the pores in the face of the masonry so that the masonry repels the water instead of absorbing it. They will not bridge cracks or voids and these should be repaired prior to treatment.

Acrylics form an elastic film over the surface of the masonry as a barrier to water. They have a limited ability to bridge small cracks or voids.

Sealers may cause more harm than good. The Brick Institute (Technical Note No. 7E[5]) describes the possible dangers with the indiscriminate use of silicones and other clear penetrating sealer solutions. They also provide a checklist of 10 points that should all be satisfied to ensure a high probability of acceptable performance. Most of the controversy surrounding clear water repellent coatings relate to their ability to breath and allow trapped moisture within the brick to escape as vapour. For more discussion, see Item 5.2.5.5, Efflorescence and Item 5.2.5.6, Freeze Thaw.

Sealers should be used with the understanding that there is some accompanying risk. It is appropriate to undertake field trials on a portion of the wall and monitor the results over a period of time. Some investigators believe a period of months is sufficient while others have suggested it may take 2 to 5 years for problems (if any) to develop.

Note that some aspects of sealer performance can be checked immediately. Improved water penetration can be checked with field measurement techniques such as the Rilem tube – see Item 2.4.7.2. Potential freeze thaw susceptibility can be checked by saw–cutting out a treated

sample of veneer for lab testing.

For additional information on sealers refer to NCMA-TEK 10B[6], Drysdale and Suter[2], BIA Technical Note 7E[5], CBD 131[7], CBD 162[8] and NCHRP Report No. 244[9].

### Colour Treatment

For discoloured bricks or inconsistent colour between old and new masonry units, corrective colour treatment can be applied. Colour matching is easier if the veneer is cleaned first – see Item 5.2.5.10, Cleaning. Colour treatments are available in variety of formulations that essentially bond pigment to the surface of the brick. These treatments may or may not contain some of the characteristics of water repellent sealers. Consult with the manufacturer.

### 5.2.5.10 Cleaning

### General

See "Masonry Cleaning – the State of the Art – STP 935" by D.W. Boyer[3].

Boyer argues that damage rendered to a masonry facade by the accumulation of surface deposits and staining goes beyond aesthetic deformation. Heavy surface deposits can contribute to the progressive decay of masonry substrates. Factors contributing to masonry decay include:
* Surface salts – see efflorescence
* Dirty wet surfaces can react with atmospheric gases (carbon dioxide, sulphur dioxide and nitrogen) to form corrosive liquids (carbonic, sulphuric and nitric acid)
* Dirty wet surfaces provide the necessary life support system for micro-vegetation which in turn enhances moisture retention and feeds on free lime components of mortar creating corrosive secretions.
* Hygroscopic staining may contribute additional moisture drawn from the surrounding atmosphere

Three basic cleaning procedures are reviewed – water, abrasive and chemical. Water includes, steam cleaning, water soaking and pressure washing. The only abrasive procedure reviewed is grit blasting which is generally not recommended. Chemical procedures include acidic cleaners, alkaline cleaners and organic solvents. Guidelines are provided for designing an effective cleaning program including job site testing and sample specification.

See also BIA Technical Note No. 20[10] where a number of cleaning techniques are recommended for various stains.

### Graffiti Removal

See "Performance Tests for Graffiti Removers – STP 935", by J.R Clifton and M. Godette.

This paper reviews the results of testing with various combinations of graffiti materials, substrates and removers.
- Graffiti material tested – aerosol paints, crayons, lipstick, felt–tip pen markers
- Substrates tested – red clay brick, Indiana limestone, Briar Hill sand-stone, aluminum
- Removers tested – 24 various cleaners and paint removers

The effectiveness of various removers is reported along with objective techniques for assessing the success of any particular process.

### Efflorescence

Gauri et al.[11] argue that the common techniques of washing with steam and water may not be effective. The water along with dissolved salts are pulled by capillary force into the deeper layers of masonry while removing some salts from the surface region only. After a period of time, the sur-face salts are likely to re–appear. Instead, they propose an experimental suction technique to be used in combination with washing that pulls the salt saturated solution from the brick.

If the source of the moisture has been eliminated, light efflorescence can be left for removal by weathering. (Note that efflorescence is normal for new masonry and will disappear on its own with weathering.)

Other effective techniques include hand scrubbing with a dry stiff brush or clear water and a stiff brush. Heavy efflorescence may require more ag-gressive proprietary type cleaners. It is imperative that the wall be satu-rated before and after the solution is applied[10].

## 5.2.6 Brick Ties – Strength, Serviceability and Building Science Issues

See also Items 3.6 and 4.3.2.5.

Inadequate brick ties can be supplemented with retrofit ties or the structural be-haviour of the wall system can be altered so that the load on any individual tie is reduced. See Item 5.2.13.2, The Strong Post Solution.

### Retrofit Brick Ties – Strength and Serviceability

Retrofit brick ties should conform to the requirements of A370-94[12], Clause 11.

The tie requirements in this clause are similar to those for new construction except that:

- The thickness of repair connectors can be greater (assuming structural requirements are met).
- The spacing of repair connectors can be greater (assuming structural requirements are met).
- The parts of a repair connector are to be made from the same material or compatible materials to control chemical or electrochemical action.
- It is recommended that repair connectors be provided with Level III corrosion protection (stainless steel or equivalent) where the existing masonry connectors have experienced a corrosion-related problem or where the existing connectors are Level III materials.
- It is recommended that existing mortars be checked for chloride ion content to assist with the selection of appropriate connector material.

A number of MV/SS retrofit brick ties are commercially available and have been extensively tested in a CMHC sponsored research program at the University of Waterloo[13, 14, 15, 16, 17]. A total of 12 tie systems were initially included in the program with 5 "finalists" selected for structural testing and building science review. The final 5 are illustrated in Figures 5 – 4 to 5 – 8*. Note that retrofit ties are divided into two subgroups – those that can be installed from the exterior and those from the interior.

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\*      *Figures 5 - 4 to 5 - 8 have been reproduced from Burnett[17].*

Fill with Epoxy
and Grout

1/2 " dia. hole in
masonry mortar joint

Chemical Connection
in Brick Veneer

Clay Brick

3/16" galvanized plated
threaded rod

Expandable Rubber
tube with Acrylic
interior in a 1/2"
dia. hole

13mm Exterior Drywall

90mm Steel Stud 20 gauge

3/4" dia. hole in
Steel Stud

Expanded Molly Nut

6 mil Polyethylene Vapour Retarder
13 mm Gypsum Board

Mechanical Connection
in Steel Stud

FIGURE 5 - 4       DINAL EXTERIOR RETROFIT MV/SS TIE
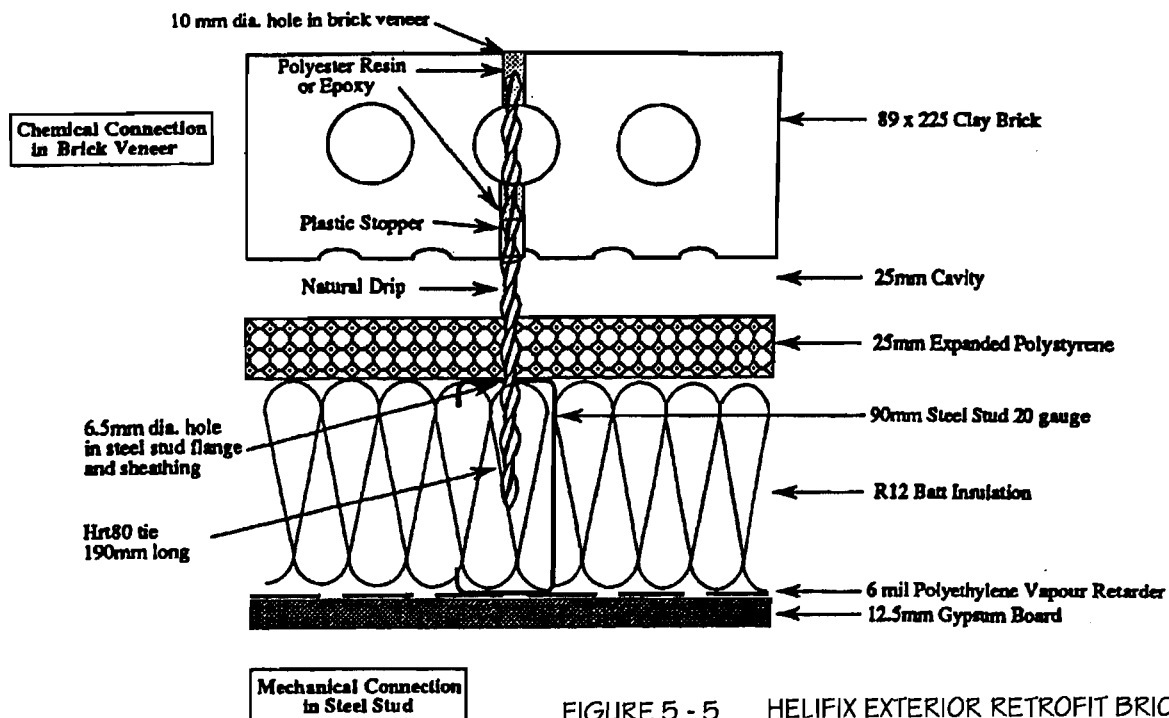
FIGURE 5 - 5     HELIFIX EXTERIOR RETROFIT BRICK TIE
(Dry fix in steel stud - polyester
resin in brick veneer)
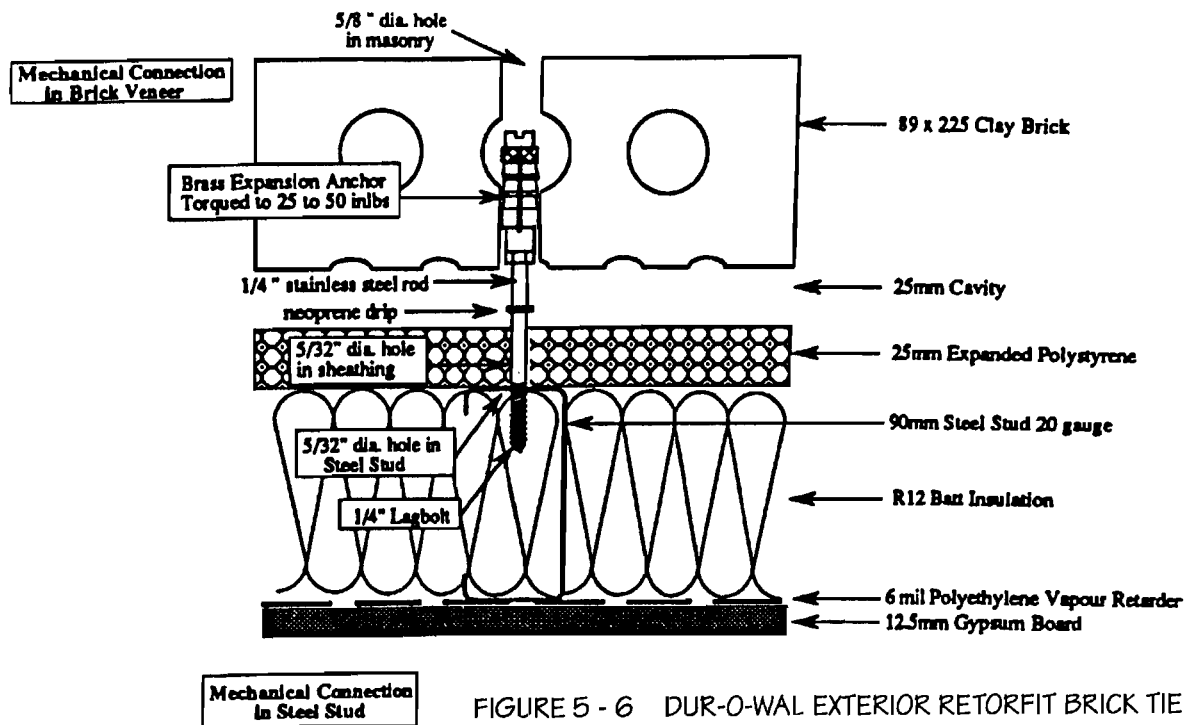


FIGURE 5 - 6     DUR-O-WAL EXTERIOR RETORFIT BRICK TIE
(Lag bolt to steel stud - expansion
anchor to brick veneer)

Mechanical Connection in Brick Veneer

6.0mm dia. hole in brick

89 x 225 Clay Brick

Natural Drip

25mm Cavity

25mm Expanded Polystyrene

6.5mm dia. hole in steel stud flange and sheathing

90mm Steel Stud 20 gauge

HRT80 Tie 220mm long

R12 Batt Insulation

6 mil Polyethylene Vapour Retarder
12.5mm Gypsum Board

Mechanical Connection in Steel Stud
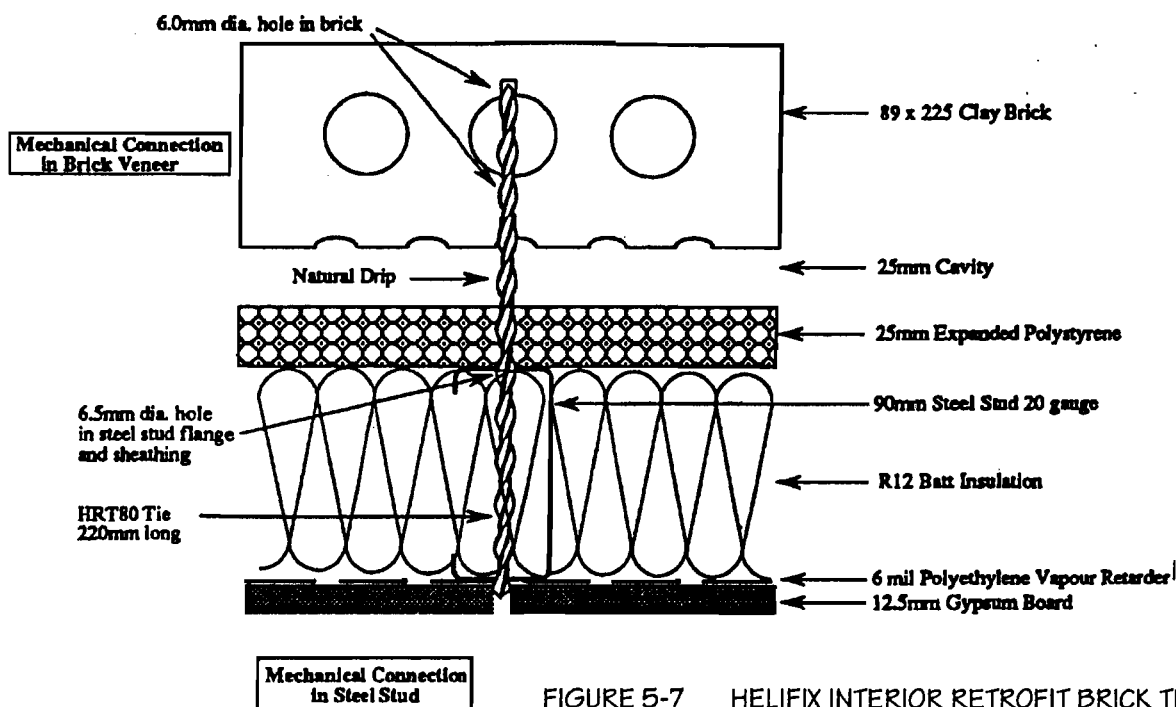
FIGURE 5-7     HELIFIX INTERIOR RETROFIT BRICK TIE
(Dry fix in steel stud and brick veneer)



Chemical Connection in Brick Veneer

89 x 225 Clay Brick

3/8 " dia. hole in masonry

Two component Epoxy

25mm Cavity

3/8" dia. hole in flange and sheathing

25mm Expanded Polystyrene

90mm Steel Stud 20 gauge

3/8" dia. x 10" long Stainless Steel Wire Mesh Screen

R12 Batt Insulation

6 mil Polyethylene Vapour Retarder
12.5mm Gypsum Board

1/4"-20 hex nut
1/4" washer

1/4" dia. x 10" long Stainless Steel Anchor Rod

3/8" dia. hole in gypsum board

Chemical Connection in Steel Stud

FIGURE 5 - 8     DUR-O-WAL INTERIOR RETROFIT BRICK TIE
(Two component epoxy fix in brick and steel stud)

The strength of the ties is summarized in Table 5 – 3.

| Table 5 - 3[7] Retrofit Tie Strength Summary (From Burnett[17]) | | | | |
|---|---|---|---|---|
| Type of Fix | Type of Tie | Steel Stud Thickness (mm) | Characteristic Strength[1], $P_M$ (kN) | Proportional Limit Load[2], $P_p$ (kN) |
| Exterior | Dinal *(9.53 mm φ holes)* | *1.52* *1.22* *0.91* *0.85* | - *2.740* *2.385* *1.535* | - *1.23* *0.80* *0.42* |
| | Helifix HRT80 *(6.5 mm φ in 1.52 mm stud)* *(3.2 mm φ in 1.22 mm stud)* *(2.4 mm φ in 0.91/0.85 mm stud)* | *1.52* *1.22* *0.91* *0.85* | *1.345* *1.065* *0.730[3]* *0.520[3]* | *0.63* *0.57* *0.20[4]* (0.46) *0.24* |
| | DUR-O-WAL *(3.97 mm φ holes)* | *1.52* *1.22* *0.91* *0.85[6]* | *3.630* *2.155* *1.670* *1.190* | *2.08* *1.22* *1.05* *0.35[4]* (0.39) |
| Interior | Helifix HRT80 *(6.5 mm φ holes)* | *1.52* *1.22[6]* *0.91[6]* *0.85[5], [6]* | *1.390* *1.105* *1.025* *0.745[3]* | *0.40[4]* (0.77) *0.42* *0.25* *0.22* |
| | DUR-O-WAL *(9.53 mm φ holes)* | *1.52* *1.22* *0.91* *0.85* | *1.235[4]* (2.63) *1.405* *1.245* *1.000* | *0.73[4]* (1.21) *1.11* *0.62* *0.62* |

*Notes to Table 5 - 1*

(1)    *The Characteristic Strength is based on the mean ultimate load of at least 5 tests less 1.5 standard deviations (as required by A370-94[12] ). Factored resistance =  $\phi\,P_M$  with  $\phi = 0.60$  for the failure mechanism observed in these tests. The factored resistance should be greater than or equal to the factored load.*

(2)    *The Proportional Limit Load,  $P_P$ , is the point at which the load-deformation curve for the tie went non-linear. Although not required by A370-94[12], Burnett[17] suggests treating this load as a serviceability limit. If this approach is adopted,  $P_P$  should be greater than the specified load.*

(3)    *These loads fail to meet the A370-94[12] provision that  $P_M$  be not less than 1.0 kN*

(4)    *Apparently anomalous because the standard deviation (variability of results) is large enough to distort  $P_M$  or  $P_P$ . Mean value is given in brackets alongside.*

(5)    *A370-94[12] requires retrofit ties to have total mechanical play less than or equal to 1.2 mm. The only tie not to meet this criterion is the Helifix Exterior for 0.85 mm stud thickness.*

(6)    *A370-94[12] requires the sum of displacement and free play for ties not to be more than 2.0 mm when tested under a compressive or tensile load of 0.45 kN. The following ties did not meet this criterion:*
       *● Helifix (Exterior and Interior) when used in studs 1.22 mm and thinner.*
       *● DUR-O-WAL (Exterior) when used in studs 0.85 mm and thinner.*

(7)    *Refer to the original reports for more detail on the structural testing[14, 16, 17].*

The following additional comments on the strength and serviceability performance of these ties should be noted:
- The performance of the Helifix ties with respect to mechanical play and stiffness disqualifies the tie for studs 1.22 mm and thinner – essentially 100% of the retrofit market. This tie could perform with thinner studs but only if some modifications are made for better connection to the stud.
- The DUR-O-WAL exterior tie performed well in the strength testing but should only be used with the understanding that the 1/4" lag bolt is a corrosion sensitive connection detail. This connection is essentially the same as a sheet metal screw in pull-out – see Item 3.6.2 for additional discussion.
- The DUR-O-WAL interior tie failed in somewhat brittle fashion and may not be the best choice where seismic performance is a consideration.
- The fire performance of ties that rely on epoxy or polyester adhesives should be confirmed if a rated assembly is required.
- These ties can also function as bridging to restrain the studs from twisting. See Item 5.2.14, Bridging.
- Although not discussed in the research reports, some of these ties may also have the potential to transfer shear in the vertical direction to create some degree of composite action (bending about a common neutral axis) between the steel stud backup and the brick veneer.

### Retrofit Ties – Building Science Issues

The building science performance of the 5 retrofit brick ties (Figures 5 – 4 to 5 – 8) was also reviewed at the University of Waterloo[15].

The potential for thermal bridging and heat loss was studied analytically using the ASHRAE zone method. As indicated by the researchers, the analysis method is approximate only, suitable for studying relative rather than absolute effects. The findings included the following:

- Thermal bridging problems with retrofit ties (cold spots on the interior wall) are reduced if exterior insulation is in place.
- The retrofit ties increase heat loss but not significantly.

Air leakage was studied experimentally with the following findings:

- If the air barrier is the interior sheathing, then an exterior fix will not increase air leakage across the wall but will decrease the air tightness downstream from the primary air barrier and may improve wall drying.
- If the air barrier is the interior sheathing and an interior fix is used, then the air barrier requires careful re-sealing at the tie location.
- If the air barrier is the exterior sheathing, then either an interior fix or an exterior fix could affect the air tightness of the wall. The ties with the least potential for leakage are the DUR–O–WAL exterior, the DUR–O–WAL interior and the Dinal exterior fix.
- The Helifix interior and the Helifix exterior fix have an impact on air leakage that is an order of magnitude greater than that of the other ties.

The potential for transporting moisture to the backup wall was reviewed with the following conclusions:

- Ties (particularly Helifix) should be installed with a slight slope downward to the outside.
- With the DUR–O–WAL interior fix, care is required to insure that the epoxy seals the hole in the exterior sheathing.
- The performance of the Dinal tie could be improved with a neoprene drip added to the rod of the tie.

Corrosion issues were reviewed with the following conclusions *(comments in italics added)*:

- Only stainless steel ties should be used for remedial work *(A370-94[12] would allow hot-dipped galvanized ties in some circumstances)*.
- The interior fix type ties are less inclined to fail due to corrosion. If the connection at one flange should corrode and fail, then resistance is still available from the connection at the other flange.
- The DUR–O–WAL exterior fix tie may be more vulnerable to corrosion because it is fabricated from dissimilar metals.
- The Dinal exterior fix tie as tested, consists of carbon steel with an *(electro-)* galvanized*)* coating which does not satisfy *(Level II or)* Level III corrosion resistance

requirements in A370-94[12]. Apparently, the components of this tie are available in stainless steel.

### Retrofit Ties – Installation Considerations

Field trials were undertaken with all of the ties except for the Dinal exterior fix. These trials are reported in the Initial Exploratory Study[13] which includes details on tie installation and evaluation.

All of the installation methods depend on accurately locating the studs. A number of recommended methods evolved from experience gained during the field trials.

To locate steel studs working from the interior:
- Identify likely positions adjacent to windows and doors
- Use drawings, if available to identify likely positions
- Use a metal detector (stud finder)
- Confirm the stud location and orientation with a hammer and a nail used as a probe through the interior drywall.

To locate studs working from the exterior:
- Identify likely positions adjacent to windows and doors
- Use drawings, if available to identify likely positions
- Working from the interior identify stud locations with a metal detector (stud finder) and transfer these location to the exterior
- Saw–cut a horizontal mortar joint and prod with a thin rod to locate studs and determine orientation.

Note that the use of a metal detector from the exterior was not successful.

### 5.2.7 Flashing

See also Item 3.7

Flashing repairs at shelf angles require openings in the brick veneer similar to those described in Figure 5 – 1.

Where possible, flashings should be installed behind exterior building paper or sheathing in a manner similar to that illustrated in Figure 3 – 15. An alternative detail is illustrated in Figure 5 – 9. See also Drysdale and Suter[2] and BIA Tech Note No. 7[24] for more information on detailing flashings.

Sealant

Continuous fastening
termination bar
fastened to steel studs

Retrofit flashing to
outside of exterior
sheathing
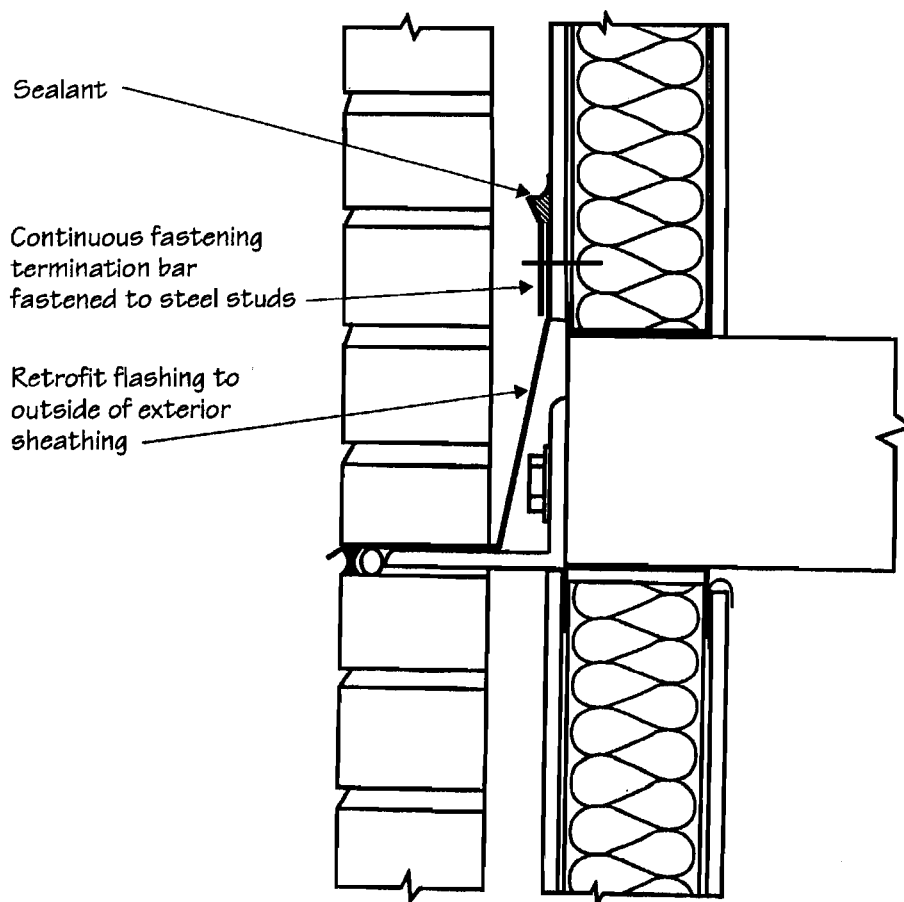
FIGURE 5 - 9     RETROFIT FLASHING DETAIL

### 5.2.8  Air Space

See also Item 3.8.

Where air spaces are large and the capacity of brick ties is compromised, retrofit ties can be installed. See Item 5.2.6.

Where air spaces are too small and/or compromised by mortar bridges, repairs are usually undertaken because of excessive water penetrating to the backup wall. The following are some of the strategies that could be implemented to reduce the amount of moisture entering the wall system from the outside:
- Re-caulk
- Install baffles in the vents to inhibit the penetration of wind-driven rain.

- Repair leaks around windows
- Apply sealer to the brick veneer locally or generally as required. See Item 5.2.5.9.
- If moisture penetration is widespread, install a complete new weather screen on the outside of the veneer (metal, EIFS, etc...)
- If moisture penetration is local, remove the brick as required, clean the cavity and re-brick.

### 5.2.9 Exterior Insulation and/or Exterior Sheathing

See also Items 3.9 and 4.3.2.6.

Exterior insulation is difficult to retrofit. Burnett and Wegner[15] reviewed the literature with respect to retrofit cavity fills using foam in place insulation materials (urethane and ureaformaldehyde). They concluded that this approach was not appropriate for MV/SS walls primarily because of moisture concerns. In testing with brick veneer and concrete block backup, the foam materials decreased the leakage rate into the cavity but increased the amount of water crossing the cavity to the backup. This strategy might be successful if the water entering the wall system from the outside is minimal. See Item 5.2.8, for some suggested strategies to reduce rain water penetration. Exterior insulation that is not built tight to the backup is also difficult to repair. For these insulation deficiencies, the most economical solution may be to install a new insulated wall system on the outside of the existing veneer.

For exterior sheathings that permit water penetration into the stud space, the amount of water entering the wall system from the outside must be reduced. See Item 5.2.8.

The repair of exterior sheathings acting as air barriers is difficult. The most expedient solution is likely to abandon the exterior sheathing as the air barrier and mobilize the interior drywall instead. See Item 5.2.10. Note that the resulting double barrier may be a concern with respect to drying for the stud space insulation. It may be prudent to introduce deliberate leaks into the exterior air barrier although this could have consequences with respect to water penetration from the outside.

### 5.2.10 Air Barriers

See also Items 3.10 and 4.3.2.7.

Exterior air barriers are inaccessible unless the brick veneer is removed and are therefore difficult to upgrade. See Item 5.2.9, Exterior Insulation and/or Exterior Sheathing for further discussion. Some exterior air barrier leaks, such as leaks in the window shim space, can be repaired with retrofit foam from the inside. See Figure 5 - 10.

Interior drywall air barriers can be upgraded by sealing identified leaks.

Based on the work by Wilson and Drysdale[18], it is likely that leakage will be observed where the drywall intersects the floor slab or intersecting shearwalls. The drywall should be caulked to the concrete floor or wall to insure the continuity of the air barrier. In addition to performance requirements, caulking materials for interior work should be paintable and odour free.

The continuity of the air barrier to the window is another common weakness. See Figure 5 – 10.



Retrofit foam for continuity of exterior air barrier with window

Retrofit sealant for continuity of interior air barrier with window
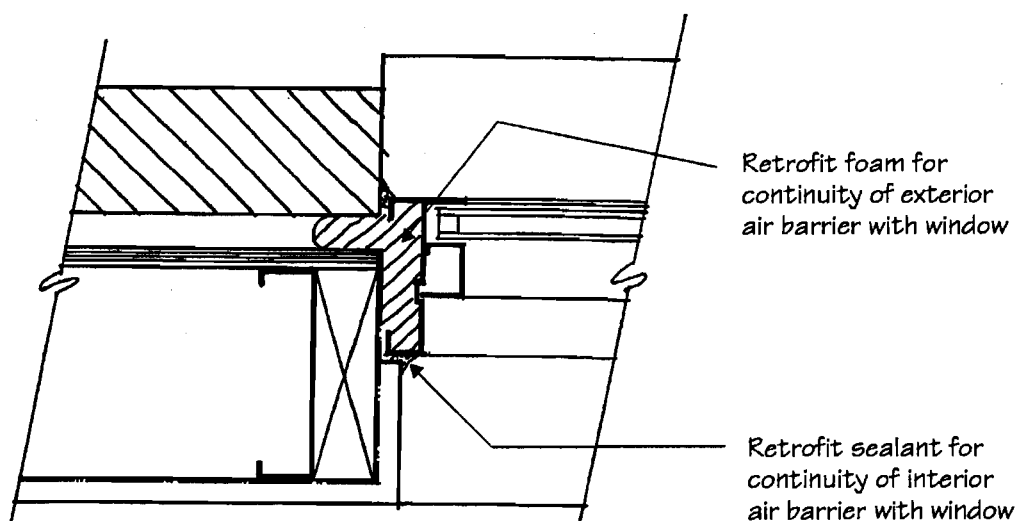
FIGURE 5 - 10        RETROFIT DETAILS FOR CONTINUITY OF INTERIOR
                     OR EXTERIOR AIR BARRIER WITH WINDOW

Intersecting drywall partitions are frequently constructed without considering the continuity of the drywall air barrier. Figure 5 – 11 illustrates a possible retrofit scheme.

Retrofit foam for continuity of
interior drywall air barrier past
intersecting partition wall
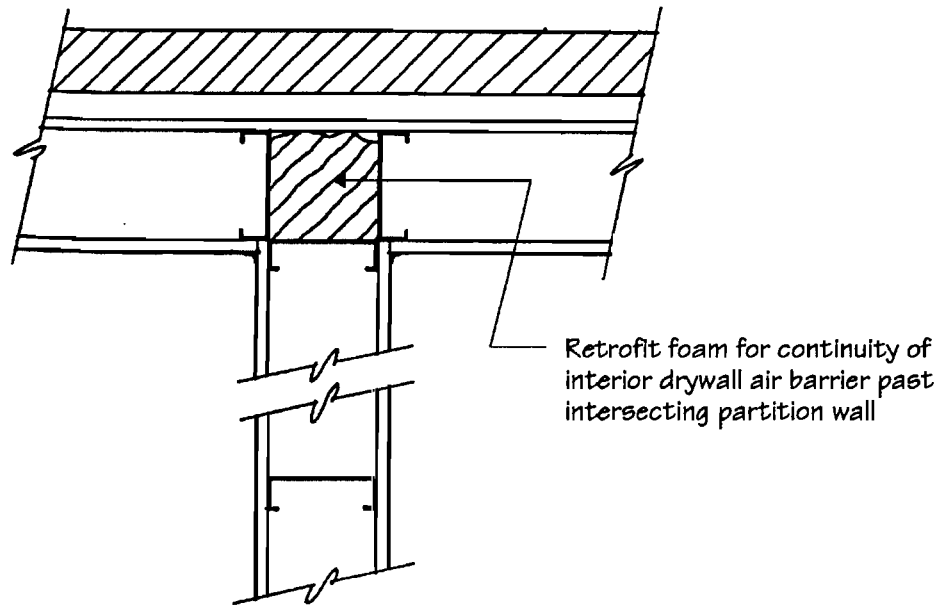
FIGURE 5 - 11     RETROFIT DETAIL FOR CONTINUITY OF INTERIOR DRYWALL
AIR BARRIER PAST INTERSECTING PARTITION WALL

Unsealed electrical outlets in exterior walls boxes on exterior walls can contribute
substantially to air leakage. The retrofit approach adopted depends on the degree
of air tightness required. Retrofit foam gaskets under the cover plate in combina-
tion with "kid-proof" plastic plug inserts can substantially reduce air leakage at
outlets but not eliminate it. Some investigators report success with foam-in-place
treatment around the electrical box. Sealing the holes in the electrical box and
sealing the junction between the box and the drywall may also be sufficient. When
sealing or foaming around electrical boxes, insure that the procedure conforms to
the requirements of the electrical codes.

Other common leak locations (behind baseboard heaters, for example) may be
more difficult to access.

Note that a tighter interior drywall air barrier will likely be subject to more wind
load than was previously the case and the bending strength and fastening should
be checked in accordance with the strength requirements in Chapter 4.

For further information on air barriers in general, see the reference list in Chapter
3 (Item 3.10.1).

### 5.2.11 Stud Space Insulation

See also Item 3.11.

Deficient stud space insulation typically does not fill the height (sagging) or width (improperly sized batts) of the stud space. If overall heat loss is a concern, supplementary insulation can be added by removing the interior drywall.

Similarly, to improve overall heat loss, stud space insulation can be added where none existed before. Note that adding this insulation may create other building science problems as a result of shifting the dew point towards the interior. See Chapter 3, Item 3.1.5.2, The Interaction Between Air/Vapour Flow and Thermal Performance.

### 5.2.12 Interior Sheathing

See also Item 3.12.

See Item 5.2.10, Air Barriers.

### 5.2.13 Typical Steel Studs

See also Items 3.13 and 4.3.2.9.

Steel studs can be reinforced a number of ways, depending on the type of over-stress that is to be corrected.

#### 5.2.13.1 *Inadequate Flexural Strength or Stiffness – Reinforced Stud Solution*

Studs with inadequate flexural strength or stiffness typically require major reinforcement.

One approach is the introduction of reinforcing studs to act in combination with the existing studs. These reinforcing studs are best installed in a back to back fashion as illustrated in Figure 5 – 12 after removing the interior drywall, stud space insulation and bridging. The spacing and factored loads for the interconnecting sheet metal screws should be determined in accordance with the requirements of S136–94[19]. Refer also to CSSBI[20], Appendix B, for sheet metal screws factored resistances. Note that two studs back–to–back (compared with single studs) are considerably more stable with respect to lateral instability and are not required to resist torsional loads. As a result, bridging may no longer be required. See S136–94[19] for guidance on bridging requirements for this type of built-up member.

This type of reinforcement also enhances the web crippling strength of the member. Back–to–back studs have more than twice the web crippling strength of the individual members – see S136–94[19].



Reinforcing stud ——————— Original stud

Sheet metal screws
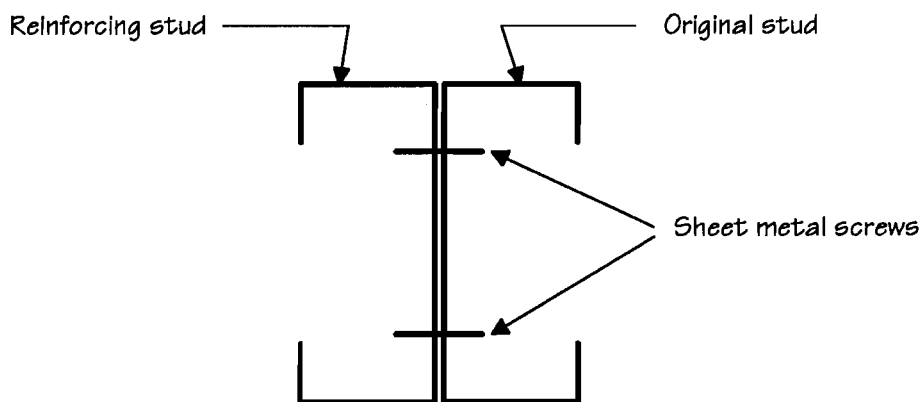
FIGURE 5 - 12    STUD REINFORCEMENT BY CREATING BACK-TO BACK
                BUILT-UP MEMBER

Another approach is to reinforce each stud with a track section as illustrated in Figure 3 - 20b. Note that this type of reinforcement is suitable for flexural stiffness and strength only. The reinforcing track does not continue into the top and bottom track to provide any additional web crippling strength. Where web crippling is an issue, a reinforcing angle can be added as illustrated in Figure 3 - 20b or as illustrated in Figure 5 - 18 where the angle is connected to the studs and to the concrete. Compared with individual studs, this type of reinforcement is also an improvement with respect to bridging requirements . Note, however, it is difficult to analyze the torsional behaviour of this type of built–up member since the member may or may not behave as a true closed section depending on the amount of interconnection.

### 5.2.13.2 *Inadequate Flexural Strength or Stiffness – Strong Post Solution*

Keller* and Trestain[21] have reported a retrofit scheme that bypasses the steel stud backup as a support for the brick veneer.

This repair was successfully carried out on two MV/SS high–rise apartment buildings where the steel studs (typical studs and jamb studs) were

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*     *Heinz Keller of Keller Engineering and Associates Inc., Ottawa Consulting Engineers was responsible for the "strong post" idea from concept through development, design and implementation.*

grossly overstressed in moment, shear, web crippling and combined bending and shear.

Strong posts consisting of telescoping pairs of HSS sections (Figures 5 – 13 and 5 – 14) were installed in both buildings with spacing between posts ranging from 1.2 – 2.5 m. The posts were attached to the concrete slab above and below with drilled expansion anchors and to the brick veneer with stainless steel bolts in epoxy sleeve anchors.

The brick was treated as a structural element, carrying 100% of the wind load, spanning horizontally between the steel posts or between the steel posts and existing concrete columns. The brick was attached to the concrete columns with Helifix retrofit exterior ties. The layout of steel posts and concrete columns for a typical portion of the wall is illustrated in Figure 5 – 15. Note that by making use of the concrete columns relatively few steel posts were required.

The steel posts were installed by removing drywall from floor to ceiling between 2 studs. The telescoping detail allowed the posts to fit over obstructions such as baseboard heaters and into the bottom track. The posts were extended and connected at the top and the bottom to the concrete slabs. The brick connectors were installed and a shim was driven into the slight gap between the inner and outer HSS posts at the bottom. See Figure 5 – 13 for the location of shims. This shimming was necessary to control the mechanical play that existed between the telescoping posts*. The original stud space insulation and the interior drywall were replaced to complete the work. The typical disruption to tenants was 5 days.

The horizontal bending strength of the brick veneer was confirmed by cutting a number of masonry specimens from the wall and testing them as illustrated in Figure 5 – 16.

With the strong post solution, the brick veneer is designed to carry 100% of the wind load, but the cavity is vented to the outside and the interior drywall air barrier is still subject to a significant portion of the wind load. This drywall air barrier applies load to the original studs which in turn apply load to the brick veneer via the original brick ties. Thus even with the strong posts in place, the studs and the ties are still required to perform structurally but with smaller load for the ties and a reduced span for the studs.

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\*   *In the absence of guidelines for acceptable mechanical play, the S304.1[22] requirements for masonry ties were adopted. For flexible structural backing systems, S304.1 specifies that deflections due to 1/2 the total mechanical play plus deformations at 0.45 kN of tension or compression be less than 1.0 mm. Analysis of the telescoping post detail indicated that shimming would reduce the total mechanical play to an acceptable range of 0.1 mm to 0.5 mm.*

With the steel stud backup acting as the structural system (before the strong posts were installed), each brick tie has a tributary area of 0.4 x (stud spacing) x (stud height) as required by S304.1[22]. With the brick veneer acting as the structural system (after the strong posts were installed), each brick tie has a smaller tributary area equal to the horizontal x the vertical tie spacing. This smaller tributary area is appropriate given the low stiffness of the studs relative to the brick veneer. Similarly, with the brick veneer acting as the structural system, the original studs only span from tie to tie instead of from slab to slab.

With the strong post approach, window areas require special consideration. The window assemblies are typically connected to the original steel stud backup such that wind loads on the window areas will be transferred to the brick veneer structural system via the original studs and ties that surround the window openings. If the original ties are not strong or stiff enough, then retrofit ties may be required. See Item 5.2.6. In addition, the horizontal bending stresses in the brick may be higher around window openings depending on how the window assembly transfers its wind load (horizontally or vertically).*

Note that the configuration of the inner and outer posts as illustrated in Figure 5 – 13 will compromise somewhat the ability of the assembly to accommodate relative deflections between the slab above and below. The inner post moves with the brick veneer and the slab below. The outer post moves with the slab above such that upper slab deflections will cause shear deformation in each of the brick connectors.** This configuration was necessary on this project in order to locate a tie near the top of the brick while maintaining sufficient length adjustment to clear baseboard heaters that were left in place. By inverting the strong post, slab deflections are better accommodated since the posts can telescope without stressing the brick connectors in shear. However, in this inverted configuration it can be difficult to locate a brick connector near the top and still retain the necessary length adjustment in the post assembly. .

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\*    *The structural behaviour of window openings can be accurately modelled using the MVSS Finite Element computer program. See Item 4.2.4.*

\*\*   *Shear deformations will reduce brick tie capacity primarily because of the bending moments that are introduced.*

122
315
315
315
315 — 2440±
315
315
315
30±
398±

FLOOR SLAB
HILTI KB 10x70 (TYP)

℄ 8x57x57, HOLE: 16mm (TYP)

HSS 76.2x76.2x4.78, L=2050mm

HARD WOOD SHIMS

HSS 63.5x63.5x3.18, L=2050mm

FLOOR SLAB
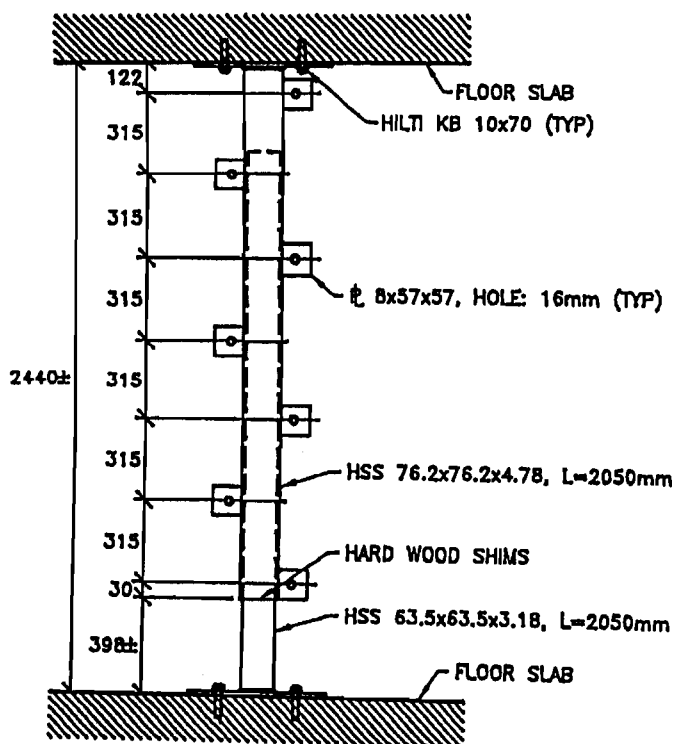
FIGURE 5 - 13

STRONG POST ELEVATION
(Reproduced from Keller and Trestain

TYPICAL EXTERIOR WALL CONSTRUCTION:
— BRICK VENEER
— AIR SPACE
— BUILDING PAPER
— EXTERIOR DRYWALL
— STEEL STUD w/ BATT INSULATION
— VAPOUR BARRIER
— INTERIOR DRYWALL

INSULATION
HILTI C-20; S.S. ROD, AISI 304
(WITH SLEEVE AND EPOXY)
SEAL

65mm

680

ALTERNATE HOLE
HILTI KB 10x70 (TYP)
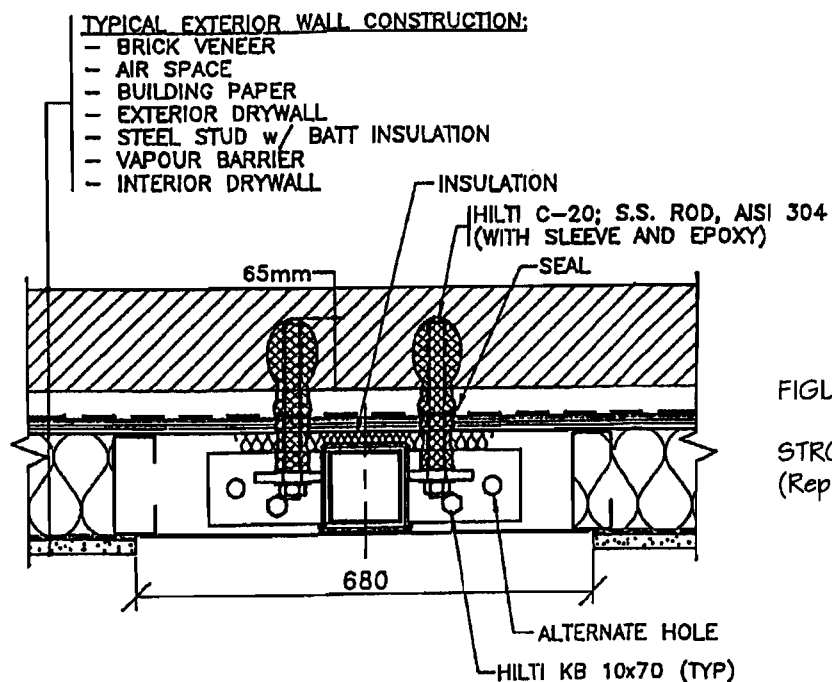
FIGURE 5 - 14

STRONG POST SECTION
(Reproduced from Keller and Trestain)

NOTES:

I - Indicates strong post to be installed from the interior

E - Indicates retrofit masonry anchors to be installed from the exterior into existing concrete columns
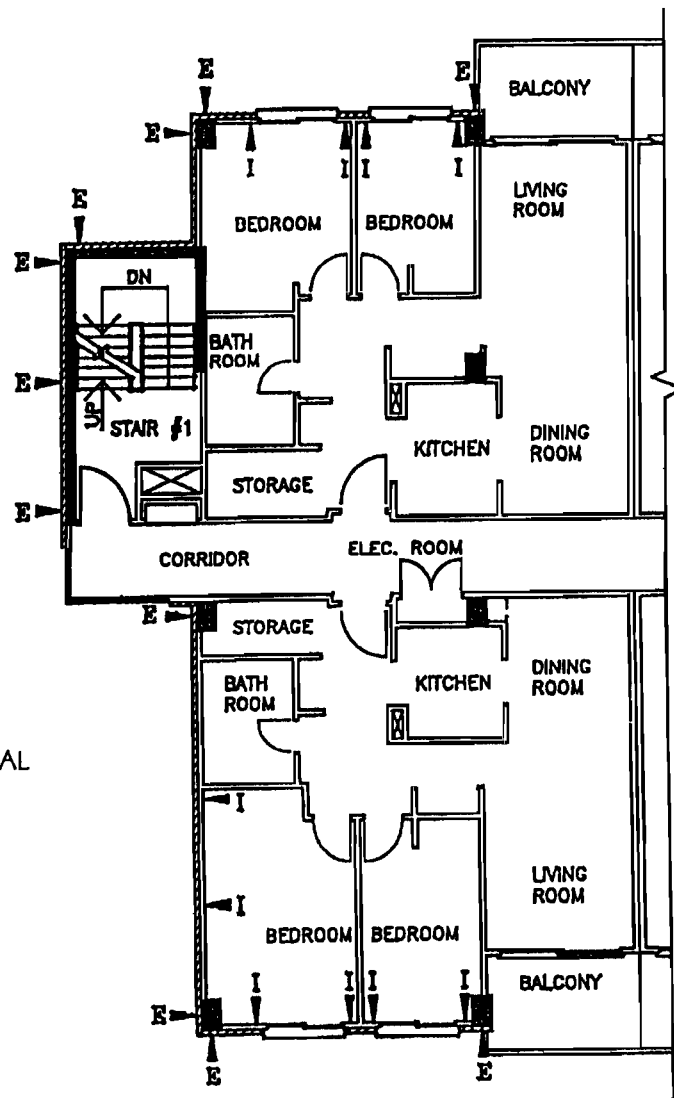
FIGURE 5 - 15

TYPICAL LAYOUT OF RETROFIT LATERAL SUPPORT FOR BRICK VENEER
(Reproduced from Keller and Trestain)
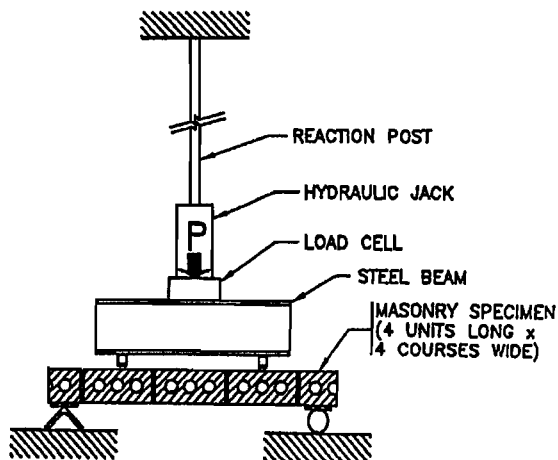
FIGURE 5 -16

APPARATUS FOR TESTING HORIZONTAL BENDING STRENGTH OF BRICK PRISMS (Reproduced from Keller and Trestain)

### 5.2.13.3 *Inadequate Web Crippling Strength*

See Figure 5 – 18 for a reinforcing detail where the web crippling strength at the end of the stud is inadequate. This detail transfers the web shear via the angle directly into the concrete and thereby eliminates bearing against the vertical leg of the track. Web crippling is, therefore, no longer an issue. Note this repair is also suitable where the track is inadequate see Item 5.2.15.

See Figure 3 – 20b for an alternative approach where the reinforcing angle is connected to the web of the stud but not to the concrete. In this case the angle behaves as a stiffener to enhance the web crippling strength of the stud. The track must have adequate capacity to resist the stud reaction. See Item 4.3.2.12 for a method to check the resistance of the track. See S136–94[19] for the design of bearing stiffeners.

### 5.2.13.4 *Missing Sheet Metal Screws – Stud to Track Connection Detail*

Missing sheet metal screws in the stud to track connection detail can be added from either the interior or exterior with cladding/sheathing removed. Sheet metal screws are typically installed through the track into the stud (see Figure 3 – 18) but can be installed in the other direction (through the stud and into the track) and perform equally well. Use short screws when installing through the stud and into the track if damage to sheathings and/or air barriers is a problem.

### 5.2.14 Bridging

See also Items 3.14 and 4.3.2.11.

A number of bridging repairs are possible depending on the nature of the deficiency.

It may be possible to bypass steel bridging and rely instead on the diaphragm strength of the inner and outer sheathings. This approach has limitations, however, particularly for gypsum drywall sheathings. See Item 3.14.2.1 for further discussion. Note also that the inner or outer sheathings may already be substantially stressed in their role as the air barrier.

For through–the–knockout bridging, inadequate connection between the bridging channel and the stud can usually be corrected by adding bridging clip angles or screws as required. See Figure 3 – 17. This approach requires removal of the interior drywall.

Periodic anchorage of through–the–knockout bridging (to the primary structure) may not be required if other aspects of the bridging are adequate. Through–the–

knockout bridging restrains the studs from rotation and only a small additional re-straining force is required to prevent the studs from translating about the weak axis. It is reasonable to assume that the interior drywall sheathing could provide this small additional restraining force. Note that this same logic does not apply to flat strap face bridging (Figure 3 – 16) where the required anchorage forces are significantly higher and may exceed the capability of the sheathings.

See also Item 5.2.13.1 for  stud reinforcement schemes that may bypass the need for bridging.

The most expedient way to retrofit bridging may be to use the retrofit ties (Figures 5–4 to 5 – 8). To some degree, these retrofit ties all have the capability to perform the role of bridging. The bridging restraint will be a function of the type of tie (ex-terior or interior) and the degree to which the diaphragm strength of the interior drywall is to be relied on. See Figure 5 – 17 and the discussion that follows.

CASE I            CASE II            CASE III
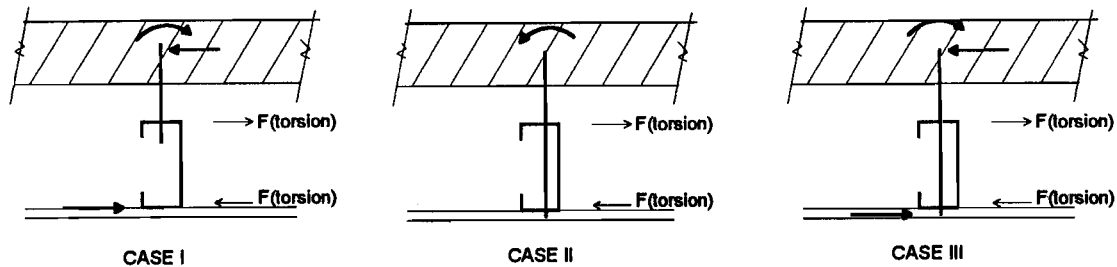
NOTES

1. F(torsion) results from the accumulated torsion at each line of bridging.
   See S136-94[19] Clause 8.3.2.4. for methods of calculating this force.

FIGURE 5 - 17    TORSIONAL RESTRAINT PROVIDED BY RETROFIT TIES

***CASE I:*** The tie cantilevers from the brick to restrain the outer stud
flange. The inner stud flange is restrained by the interior drywall dia-
phragm. This case would apply to the exterior retrofit ties where the dia-
phragm strength and stiffness of the interior drywall could be relied on.

***CASE II:*** The tie cantilevers from the brick to restrain the inner and the
outer stud flange. Any restraint from the interior drywall is neglected. This
case would apply to the interior retrofit ties where the diaphragm strength
and stiffness of the inner drywall was deemed unreliable.

***CASE III:*** The tie behaves as a propped cantilever spanning from the
brick to the interior drywall diaphragm. The case would apply to the inte-
rior retrofit ties where the diaphragm strength and stiffness of the interior
drywall could be relied on but not to the same extent as in Case I.

Note that none of these three cases relies on the outer sheathing dia-
phragm strength. Note also that the ability of these retrofit ties to perform
as bridging has not been confirmed experimentally. The factored load on
each tie (acting as bridging) can be calculated with reasonable certainty
using S136-94[19] Clause 8.3.2.4. The factored resistance for each tie
(acting as bridging) will require analytical treatment. Engineering judge-
ment is required.

### 5.2.15 Bottom Track

See also Items 3.15 and 4.3.2.12.

Track reinforcement typically involves bypassing the track as a structural element. This can be accomplished with the detail illustrated in Figure 5 – 18. The clip angle detail transfers the end shear from the stud directly into the concrete. If properly sized, the clip angle can also provide the necessary torsional restraint for the end of the stud.
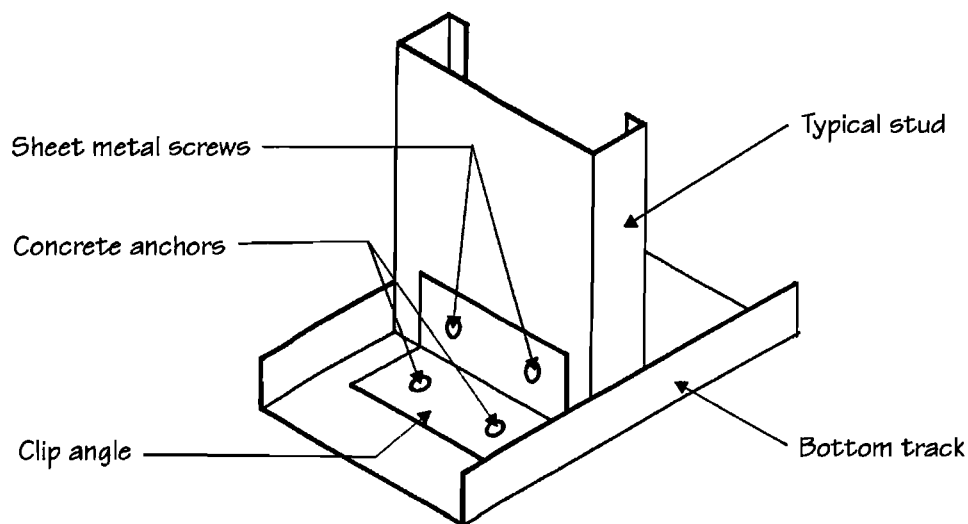


FIGURE 5 - 18        RETROFIT DETAIL FOR STUD TO TRACK CONNECTION

### 5.2.16 Top Track Assembly

See also Items 3.16 and 4.3.2.13.

Top track details can be reinforced a number of ways, depending on the type of overstress that is to be corrected.

Two retrofit details are suggested where the outer top track has inadequate strength. Both of these details essentially bypass the outer top track as a structural element.

In Figure 5 – 19, the reaction from the studs is resisted by an adhesive anchor

cantilevering from the slab above. The spacing of the adhesive anchors would be governed by either the anchor strength or the beam strength and stiffness of the inner top track. The adhesive anchor is installed through a large diameter hole cut in the inner top track. A plate with a hole size to match the anchor diameter is then welded in place to insure a minimum of mechanical play for the connection. Note that considerable care is required with respect to fire and fumes when welding from the interior side.

Adhesive anchor rod threaded
one end and plain the other

Outer top track

Concrete slab

Inner top track

Plate with hole size to match adhesive anchor rod diameter. Field weld plate to inner top track.

Opening in inner top track to facilitate installation of adhesive anchor.
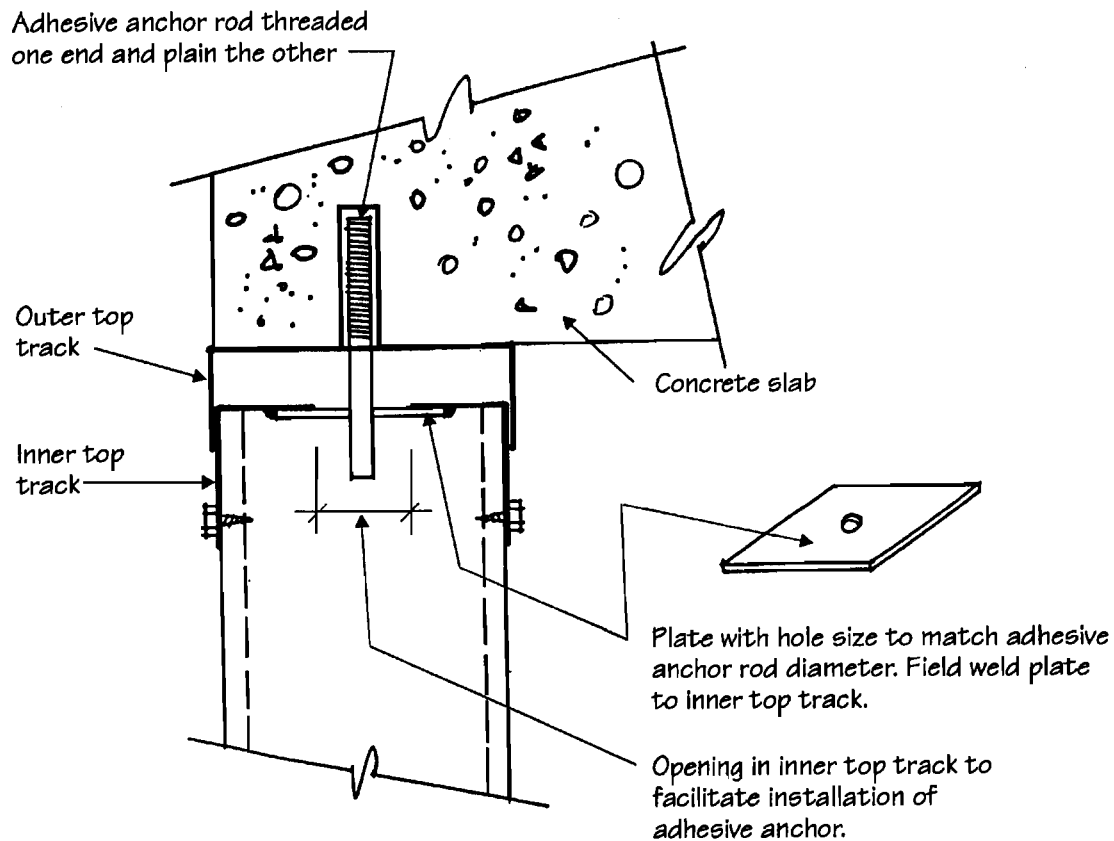
FIGURE 5 - 19　　OUTER TOP TRACK ADHESIVE ANCHOR
　　　　　　　　　RETROFIT DETAIL

Figure 5 - 20 illustrates a similar connection detail but without the welding. In this case, a wedge type drilled-in expansion anchor is used in combination with a short piece of tubing. The tubing or sleeve is in essence a very thick washer that resists the pre-tension in the expansion anchor and acts together with the anchor as a cantilever to resist the reaction from the studs. Again, the spacing of the anchors would be governed by either the anchor strength or the beam strength and stiffness of the inner top track. The anchor is installed through a hole in the inner top track that matches the outer diameter of the sleeve. Although there is no welding, the drilling can be difficult. Oversized holes would not be desirable because of excessive mechanical play.

Drilled expansion
anchor

Outer top
track

Inner top
track

Concrete slab

Cut hole in inner
top track to match sleeve
diameter

Sleeve
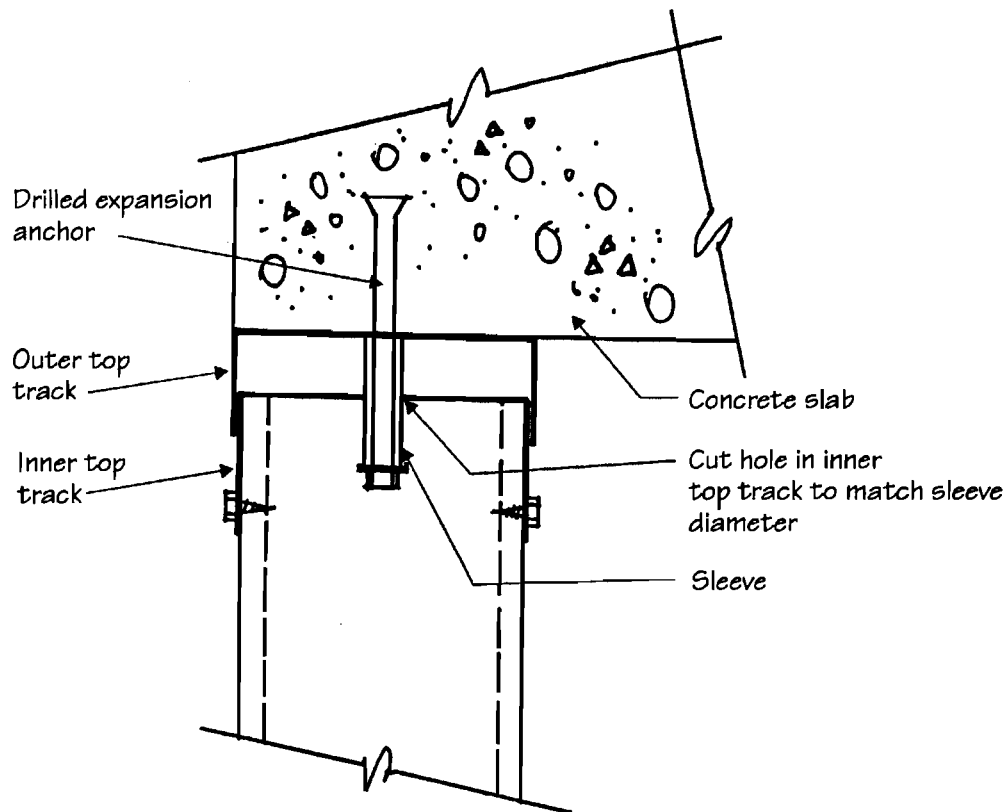
FIGURE 5 - 20          OUTER TOP TRACK EXPANSION ANCHOR
                      RETROFIT DETAIL

The retrofit detail illustrated in Figure 5 – 21 would be appropriate where the strength of the outer top track is adequate but its connection to the concrete is not. This detail can only be installed from the exterior side through an opening in the brick veneer. The brick and/or the stiffener may require cutting to insure adequate clearance – otherwise, the performance of the soft joint may be compromised.

For negative wind pressures the inner top track bears directly on the stiffeners. For positive wind pressures, the inner top track bears on the vertical leg of the outer top track on the interior side and the stiffener resists the tendency of the outer top track to slide and to overturn. Where repairs must be made from the interior side to correct for inadequate fastening to the concrete, use the details illustrated in Figures 5 – 19 and 5 – 20.

Shelf angle

Stiffener welded to
shelf angle and to
outer top track

Concrete slab

Outer top track
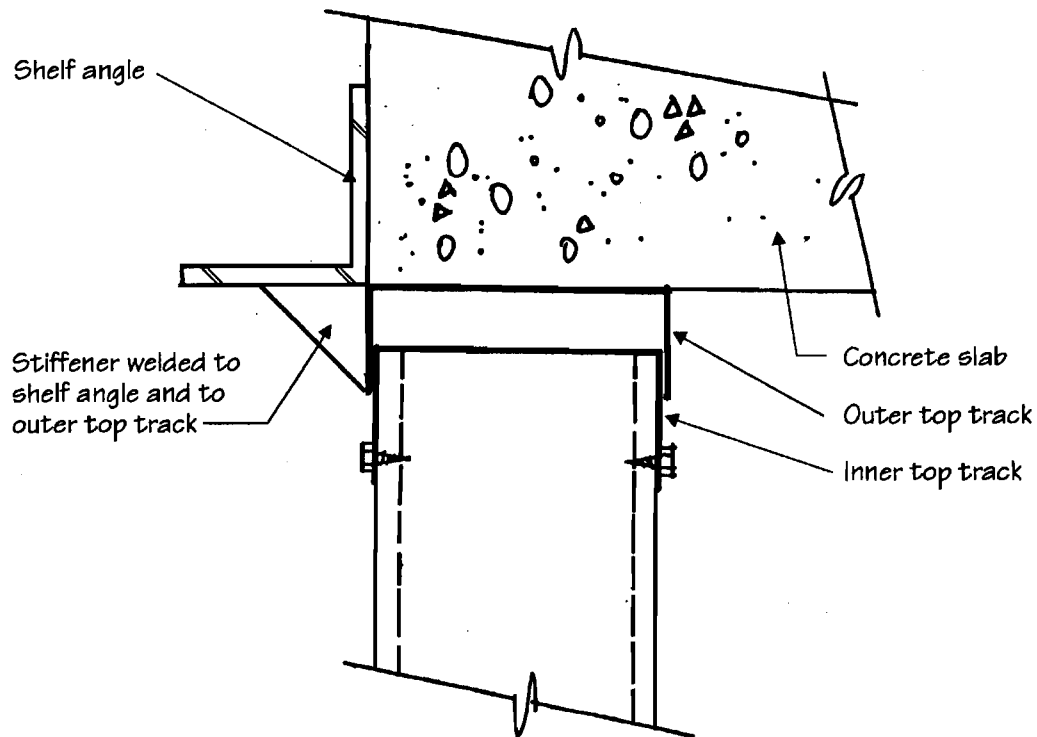
Inner top track

FIGURE 5 - 21          OUTER TOP TRACK WELDED STIFFENER
RETROFIT DETAIL

## 5.2.17 Built-up Jamb Studs

See also 3.17 (typical details Figure 3 - 20) and 4.3.2.14.

Built-up jamb studs can be reinforced a number of ways, depending on the type of overstress that is to be corrected.

For toe to toe studs that are inadequately interconnected, an expedient repair is to install retrofit brick ties to both studs (see Item 5.2.6). The retrofit ties will mobilize the brick veneer to serve as the connecting element to insure that the 2 studs work together. This repair solution is explored in the separate MVSS User Reference Manual[23], Case Studies 2 - A and 2 - B.

Built-up jambs with inadequate flexural strength or stiffness are difficult to rein-force because of interference from the window mullion and the head and sill members that frame in. A reinforcing track can be added (see Figure 3 - 20b) to

the outside stud in a toe to toe configuration. Where a significant increase in strength or stiffness is required, the strong post solution (see Item 5.2.13.2) with the reinforcing posts set beside the overstressed jambs may be appropriate.

Other reinforcement details are similar to those for typical studs. See Item 5.2.13.

### 5.2.18 Head and Sill Members

See also Items 3.18 and 4.3.2.15.

Head and sill members are difficult to reinforce because of interference from the window on one side and the jack studs on the other. For head members, insuring good tie to the brick and thereby mobilizing the horizontal bending strength of the loose angle lintel may be sufficient.

For sill members, it may be possible to add reinforcement in the depth of the air space or the veneer. An angle reinforcement, similar to an upside down loose angle lintel, might be appropriate on top of the brick at the sill level.

### 5.2.19 Parapets

See also Items 3.19 and 4.3.2.16.

Repairs to the parapet veneer are similar to the requirements for the typical floor to floor veneer.

For steel stud problems, repair usually means replacement. Refer to the CSSBI[20] for appropriate structural systems for parapets.

Note that the disruption to the water proofing membrane and flashings on the roof can make parapet repairs/replacement difficult and expensive.

### 5.2.20 Corrosion

See also Items 3.21 and 4.3.2.18.

For steel parts weakened by corrosion, reinforcement schemes will be similar to those described previously for individual wall elements.

For steel parts with no significant strength loss but whose durability has been compromised by corrosion, the rate of deterioration can be reduced by improving the building science performance of the wall (as dictated by the diagnosis).

Zinc rich paint can also be used to slow the rate of deterioration of corroded steel parts. Note that some specifications require the equivalent of a sand blasted finish

before the application of zinc rich paint. This type of surface preparation is im-
practical in the field and with light gauge material. Power wire brushing is usually
considered adequate. Note that commercially available zinc rich paints varying
considerably in zinc content. Higher zinc contents provide superior protection.

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# *Appendix B – Terminology*

Masonry (or brick) veneer

Mortar joint

Water barrier

Air space

Flashing

Weep holes (open head joints usually at 600 mm o.c.)

Sealant and backer rod

Vent holes (open head joints)

Wire pintle (or tie)

Exterior sheathing and/or rigid insulation

Batt insulation

Interior sheathing (drywall)

Vapour retarder

Bottom track

Cast-in-place adjustable anchor and shelf angle

Slab

Inner and outer top track deflection detail

Brick connector (or tie)

FIGURE B-1
TYPICAL SHELF ANGLE DETAIL

Typical (wind bearing) stud

Head track

Floor slab

Outer top track

Inner top track

Bridging clip angle

Through-the-knockout style bridging (bridging channel)

Jamb studs (toe to toe)

Web cut-out (or knockout or perforation)

Sill track

Jack stud

Bottom track
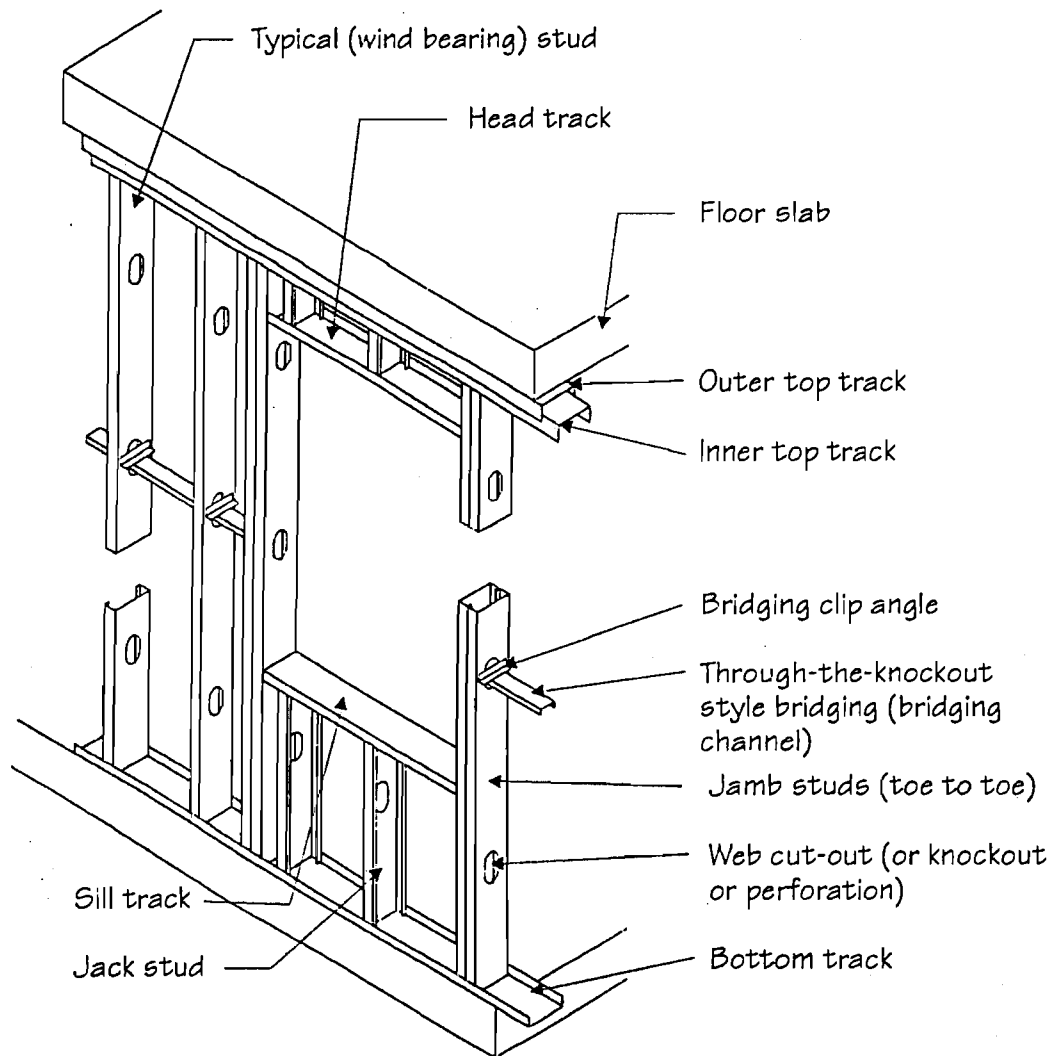
FIGURE B-2
TYPICAL STEEL STUD BACK-UP WITH WINDOW OPENING

Flange

Lip

Inside bend radius

Web
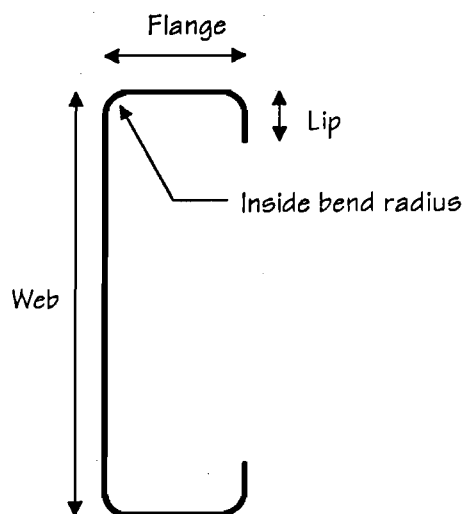
FIGURE B-3
TYPICAL STUD CROSS SECTION GEOMETRY

Flange

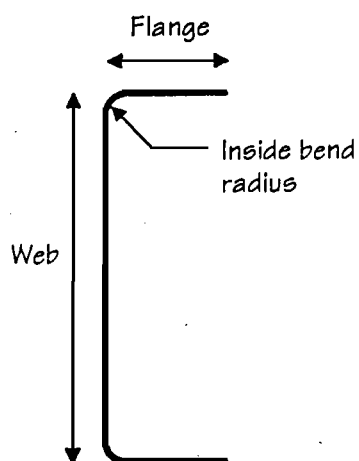Inside bend radius

Web

FIGURE B-4
TYPICAL TRACK CROSS SECTION GEOMETRY

# *Appendix C – Wall Performance Checklist*

A well designed MVSS wall should conform to the following:

## 1. Brick Veneer

### 1.1 Shelf angle

Adequate size, strength and stiffness
Adequate connection to structure
Continuous *(particularly at corners)*
Adequate corrosion protection
Minimum contribution to thermal bridging *( no continuous bridge with top track)*

### 1.2 Loose Angle Lintels

Adequate size, strength and stiffness
Adequate corrosion protection
Adequate bearing *(at lintel ends)*
No moisture accumulation

### 1.3 Expansion, Control Joints

Horizontal *(beneath shelf angles)* – Adequate size, appropriate sealants
Vertical     – Adequate size and frequency, appropriate location and sealants

### 1.4 Brick and Mortar

Brick in good condition *(no spalling, cracking, efflorescence, out-of-straightness)*
Adequate durability of brick and mortar
Adequate horizontal bending strength and stiffness *(if considered structural)*
Adequate vertical bending strength and stiffness *(if considered structural)*
Weepers – Appropriate size and spacing *(for drainage and for pressure equalization)*, No Obstructions
Vents – Appropriate size, spacing *(for wall drying and for pressure equalization)*, no obstructions, louvered screen
Limited brick overhang on foundation wall, shelf angles, lintels
Adequate damp-proofing
Adequate clearance from grade
Mortar in good condition
No inappropriate mortar additives *(e.g. chlorides)*
Well filled mortar joints *(especially head joints)*

### 1.5 Brick Ties

Adequate spacing *(horizontal and vertical)*
Appropriate type
Adequate mortar embedment

Adequate connection to steel studs *(direct steel to steel connection)*
Adequate strength and stiffness
Adequate corrosion resistance
Minimal accumulated mortar droppings

### 1.6 Flashings

Located where required *(foundation, shelf angle window sill, window head)*
Appropriate type
Continuous *(well constructed lap joints)*
Adequate end dams *(at all terminations)*
Adequate drip *(beyond face of veneer)*
Adequate durability *(during construction and in the completed wall)*

## 2. Air Space

Adequate size *(1" minimum, 2" preferred)*
Minimum number of mortar bridges
Minimum mortar build-up at shelf angle

## 3. Exterior Insulation and/or Exterior Sheathing

Appropriate type
Adequate moisture resistance
Adequate thickness, R Value
Appropriate detailing if acting as the air barrier *(continuous, low air flow properties with adequate strength, fastening, stiffness and durability)*
Appropriate detailing if not acting as the air barrier *(no unanticipated pressure difference across the sheathing and/or insulation due to wind and sufficient permeability to allow drying of the stud space insulation)*
Appropriate detailing if acting as the water barrier
Appropriate detailing if acting as the vapour barrier
Adequate strength, stiffness and durability If acting as a structural brace for studs
Adequate connection to steel studs
No convection space between insulation and sheathing
Minimal thermal bridging

## 4. Air Barrier

Appropriate type and location
Adequate air tightness of material, joints, interfaces with other building elements
Adequate strength, stiffness and durability
Adequate resistance to creep
Adequate resistance to moisture
Adequate resistance to thermal stress, building movements

## 5. Vapour Retarder

Appropriate type and location
Adequate vapour impermeability of material, joints, interfaces with other building elements

## 6. Stud Space Insulation

Appropriate type and thickness
Tight to interior and/or exterior sheathings
Tight fit to inside of studs
No sagging
Inside of built-up box members
Minimal thermal bridging
Dew point outside of stud space insulation (preferred)

## 7. Interior Sheathing

Appropriate type and detailing if acting as the air barrier *(continuous, low air flow properties with adequate strength, fastening, stiffness and durability)*
Adequate strength, stiffness and durability If acting as a structural brace for studs
Adequate connection to steel studs
Appropriate type and detailing acting as the vapour barrier

## 8. Steel Stud Back-up

### 8.1 Typical Stud

Adequate strength and stiffness
Adequate corrosion resistance
Appropriate cut-out locations *(relative to top and bottom connections, brick ties)*

### 8.2 Bridging (through the knockout, face bridging or sheathing)

Adequate strength, stiffness and connection to studs
Adequate spacing
Continuous
Adequate anchorage to primary structure

### 8.3 Bottom Track

Adequate connection to floor slab *(fastener type, size, spacing, corrosion resistance, edge distance, location relative to typical studs and built-up studs)*
Adequate strength and stiffness
Adequate corrosion resistance
Appropriate end gap stud to track *(4 mm maximum)*
Drainage capability if required by design
Continuous

## 8.4 Top Track Assembly

Appropriate detailing to accommodate slab deflections
Minimum engagement between inner and outer top track *(or between stud and track)*
Adequate connection to soffit of floor slab *(fastener type, size, spacing, corrosion resistance, edge distance, location relative to typical studs and built-up studs)*
Adequate strength and stiffness
Adequate corrosion resistance
Appropriate end gap stud to track *(for inner and outer top track detail - 4 mm maximum))*
Continuous
Adequate racking resistance in the plane of the wall *(for inner and outer top track detail)*

## 8.5 Built-up Jamb Studs

Adequate strength and stiffness
Adequate corrosion resistance
Adequate interconnection of built-up members
Appropriate cut-out locations *(relative to top and bottom connections, brick ties, head and sill member connections)*

## 8.6 Head and Sill Members

Adequate strength and stiffness
Adequate corrosion resistance
Adequate interconnection of members if built-up
Adequate connection to jamb studs

# 9. Special Conditions

## 9.1 Windows and Doors

Adequate deflection gap with brick
Adequate deflection gap with studs *(where no deflection gap detail is provided at the top track detail)*
No damage to steel stud back-up *(due to shimming and racking)* during installation of windows or doors
Adequate anchorage *(for wind and gravity load)*

## 9.2 Parapets

Adequate cap flashings
Adequate connection to roof flashings
Vertical control joints *(adequate size and frequency, appropriate location and sealants)*

# *Appendix D - MVSS Finite Element Computer Program Helpful Hints*

## 1.  MVSS Finite Element Program Start-up Tips

1.1    Set up two sub-directories as recommended in the manual and on the *read.me* file. Disk 1 of 3. For example:

```
C: ─┐
    │
    └─ MVSS ┬──────── SOURCE
            │
            └──────── RUN&DATA
```
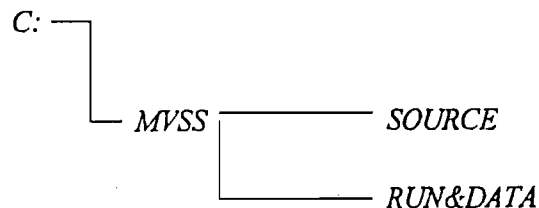
1.2    *SOURCE* – contains source programs including MVS.EXE

*RUN&DATA* – contains data files and *MVSS.BAT* and *CONFIG.DAT* both created by *INSTALL.EXE*

The program is run by executing *C:/MVSS/RUN&DATA/MVSS.BAT*.

1.3    Note that the *INSTALL.EXE* program may not work properly because it may be necessary to be in *C:\MVSS\SOURCE* before going to *A:* drive and executing *INSTALL.EXE*.

1.4    The *PKUNZIP* utility is specific for each case study – for example, *PKUNZIP* for Case 1-A may not work for Case 2-A.

## 2.  MVSS Finite Element Program General Tips

2.1    In order to run the program successfully, it is important to free as which conventional memory as possible . Note, however, the 627K recommended in the User Reference Manual is not always necessary. Because of these memory limitations, the MVSS program should be run out of *DOS*, not *Windows* – at least when running the analysis part of the program.

Errors such as "Cannot open error file F77L.EER" are nearly always due to insufficient conventional memory.

The available conventional memory can be checked with the *DOS* command *mem/c* and is shown as the "Largest Executable Program Size" .

2.2    In order to conserve space in conventional memory the print to file function (for printing screen graphical displays) has been disabled. The ability to send the screen graphics directly to a printer has been retained.

2.3    For wall configurations with complicated geometries, some of the graphical displays to the screen can be "too busy" and difficult to read. (For an example of this problem, see the input geometry screens for Case Study 2 – A.) Sometimes this difficulty can be overcome by erasing information of one colour to better observe the remaining information.  After the program has been run in *DOS* and the output files created and saved, run the program in *Windows* and bring up the required graphical display on screen. Transfer this display to the *Clipboard* (*Print Screen*) and then *Paste* into *Paintbrush*. Use the colour erasure function in *Paintbrush* with the foreground (left mouse button) set equal to the colour to erase and the background (right mouse button) set to black. The image can also be enlarged for better viewing.

2.4    *CONFIG.DAT* can be edited – the first line is the subdirectory where the source program *MVS.EXE* is stored and the second line contains the subdirectory for the run program (*MVSS.BAT*) and the data.

2.5    The node numbering sequence in *case1a.out* for Elastic Deformations is incorrect (1, 2, 3, 4, 7, 8, ...) in both the User Reference Manual (Page 77 – 79) and on diskette. The correct sequence is 1 – 119 with no gaps. Re-running Case 1–A will yield the correct node numbers.

       This item is addressed in the *read.me* file on Disk 1 of 3 Item 5A.

2.6    The Boundary conditions are defined by the Boundary screens – with free or fixed for x, y, z, x-rotation and y-rotation. These same inputs can be checked in the *\*.OUT* file under Nodal Configuration. The nodes are not key points but the node numbering that is generated from the key points by the program. The node numbering can be viewed with the plot routines provided. Note that the *\*.OUT* file has the x-rotaion and y-rotation columns reversed compared with the Boundary screens.

       Note also that Case 2–A and Case 2–B boundary for the stud adjacent to the shearwall leaves the masonry free but fixes the stud to the shearwall.

2.7    To enter boundary "Constraint for Node" use the F5 key. When reloading this same boundary screen the node will not appear until its number is entered followed by the return key. Note that more than one constraint can be entered for the same screen.

       Currently for Cases 1A and 1B the constraint for node number 15 is not part of the input data on diskette – only on hard copy. (Node 15 has the correct boundary conditions without explicitly declaring them. For some examples, this may not be the case)

2.8    During execution, the program makes extensive use of spill files. These spill files
       are created in the *RUN&DATA* subdirectory.

       If there is sufficient RAM available, the program will run considerably faster if the
       *RUN&DATA* subdirectory is set up on a RAM drive instead of a hard drive. Note
       that for Case 2 – A, a 10 megabyte RAM drive would be required.