# SEMINAR

# ON

# BRICK VENEER WALL SYSTEMS

PRESENTED BY:


| | |
|---|---|
| Jacques Rousseau | CMHC |
| Heinz Keller | Suter Keller Inc. |
| Robert Drysdale | McMaster University |
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LOCATIONS:

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| Calgary | June 12, 1989 |
| Winnipeg | June 14, 1989 |
| Toronto | June 16, 1989 |

SPONSORED BY:

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| CMHC | Canada Mortgage and Housing Corporation |
| CCRB | Canadian Construction Research Board |
| ABEC | Alberta Building Envelope Council |
| MBEC | Manitoba Building Envelope Council |
| OBEC | Ontario Building Evelope Council |

# TABLE OF CONTENTS

PART 1

# EXISTING DESIGN AND CONSTRUCTION CONDITIONS

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## Chapter 1 NEEDS STUDY


ABSTRACT

During the last decade, brick veneer/steel stud (BV/SS) walls have become widely used in Canada, as an economical building enclosure system in residential and commercial structures. However, the construction of these walls has preceded the development of adequate design, construction and inspection standards. This situation has led to concern over the longterm safety, serviceability and durability of this form of construction. This study gathers information related to design, construction and inspection aspects of BV/SS walls in Canada.

An industry survey was conducted in order to gather broadbased information and opinions regarding practices and concerns from people involved with the BV/SS wall system. This report summarizes the results of the survey and, based on the findings, identifies and establishes the state of industry concern.

The findings indicate that a variety of design, construction and inspection practices are employed across Canada. A need exists for some degree of formalization and standardization of the existing practices for the design, construction and inspection of the BV/SS wall system. The need for more research, education and better technology transfer is shared by the majority of respondents. The issue of corrosion of steel stud, ties and screw connections as well as steel stud flexibility have been expressed as major concerns.

## 1.0 INTRODUCTION

Brick veneer/steel stud (BV/SS) walls have become a popular and economical way to enclose a modern structure. The system is used in lowrise and mid- to highrise buildings in residential, commercial, office, institutional and industrial applications. The development of the BV/SS wall system has been encouraged by a number of factors, including::

- **Savings in Construction Costs**
  Costs less than other systems such as curtain walls or conventional masonry veneer/back-up walls

- **Economy of Space**
  Occupies less space than conventional masonry walls

- **Aesthetics**
  Can satisfy current architectural tastes towards exterior brick walls

- **Reduction in Dead Loads**
  Requires lighter structural framing and foundations

- **Construction Time Reduction**
  Allows the building to be closed in faster than with conventional masonry

- **Fire Resistance**
  Meets the NBC and other regional building code requirements.

Various opinions and practices regarding the design, construction and inspection of the system are in place as they preceded the development of a design code as well as construction standard requirements. Furthermore, problems related to its structural behaviour and the rain screen principle are not yet fully understood.

This report deals with a survey of current practices for design, construction and inspection of the BV/SS wall system in Canada. In general terms, the survey collected information about materials, design and construction issues.

The information was gathered from the following four groups:

Group A: Architects and Engineers
Group B: Masonry Contractors
Group C: Steel Stud and Drywall Contractors
Group D: Building Officials and Inspectors

Table 1 provides the statistics with reference to the number of questionnaires sent to each group. The questionnaires were tailored to each of the four groups.

Table 1: General Statistics with Reference to the Number of Questionnaires

| Group | Number of questionnaires sent | Number of questionnaires answered |
|---|---|---|
| A | 306 | 111 |
| B | 82 | 23 |
| C | 41 | 16 |
| D | 21 | 10 |
| Total | 484 | 160 |

The objective of this study is to determine the state of the art of the BV/SS wall system in Canada.

Prior to mailing the final version of the questionnaires, a pilot test run was carried out in the Ottawa area to ascertain that the questions were pertinent and understandable. An attempt was made to eliminate any bias in the sampling procedure and to collect a representative sample which describes the actual BV/SS wall situation in all regions of Canada.

In reviewing the survey findings, it should be recognized that there are limitations to a survey. The quality of a survey depends to a great extent on:

- population sample
- number of responses
- clarity of questions

- respondents' interpretation of questions
- interpretation of answers.

While the questionnaires were somewhat lengthy and detailed, respondents appear to have made an effort to answer the questions as completely as possible.

## 2.0 SAMPLING METHODOLOGY AND NATURE OF SAMPLE

The population for sampling was defined as the people and businesses in Canada involved with BV/SS walls. This population was divided into four groups as outlined in Section 1. Individuals in these groups are operating a business and/or supply service in one of the following four regions: (1) Atlantic Canada, (2) Southern Quebec and Ontario, (3) Prairie Region and Northern Ontario, (4) Pacific Canada.

The selection of individuals to be part of the survey was based on one or more of the following criteria:

- The firm or individual was found to have some experience with BV/SS walls after an interview.

- The firm or individual was referred to us by people knowing they had some experience with BV/SS walls.

- The firm or individual was listed by a construction trade association (eg. masonry, drywall, etc.).

- A number of firms and individuals were selected using random numbers and lists of names available to the public.

Fig. 1 indicates some general statistics on respondents and their location across Canada.

As shown in Fig. 1, the majority of people surveyed are in the architectural and structural design business and are located in the Southern Quebec and Ontario region.

Of the 160 firms and individuals responding to the survey, 39 (24%) did not have any experience with BV/SS walls. The balance of 121 (76%) had some degree of experience.

The respondents appear to represent collectively, BV/SS wall system experience on over 1,000 buildings. Of these, 42% were residential and 58% were commercial or industrial.

Fig. 2 illustrates some general statistics on building height, type of building system and number of years the BV/SS wall system has been employed in Canada. The survey suggests that the BV/SS wall system is more popular in buildings less than 4 storeys in height.

The statistics in Fig. 2 indicate that the BV/SS wall system has been used in Canada for approximately 8 years. While the system has been used for over 9 years in the Prairies and Northern Ontario, the Pacific region has employed it only for about 6.5 years.

## 3.0 RESPONSES

Respondents can be grouped into four categories according to their position with respect to the BV/SS wall system. The percentages shown in Fig. 3 are estimates derived from the respondents' comments in the questionnaires:

- Enthusiastic believers and users (mostly contractors)
- Users with some reservations
- Non-Users waiting for more knowledge and refinements on the system
- Non-Believers.

Twelve firms stated that they will simply not use the system, as they consider it to be unsafe.

Pertaining to the Non-Believers, their major concerns are as follows:

- Stiffness incompatibility between the steel stud wall and the brick veneer

- Unjustified reliance, implied in some designs and construction techniques, on the structural integrity of the gypsum wallboard as a structural material or as a spacer between the studs and the ties. What happens when the gypsum gets wet?

- Corrosion of the widely used screw connectors as they perforate the zinc coating of the studs

- Difficulty in providing an adequate air/vapour barrier for this type of construction and the resultant risk of

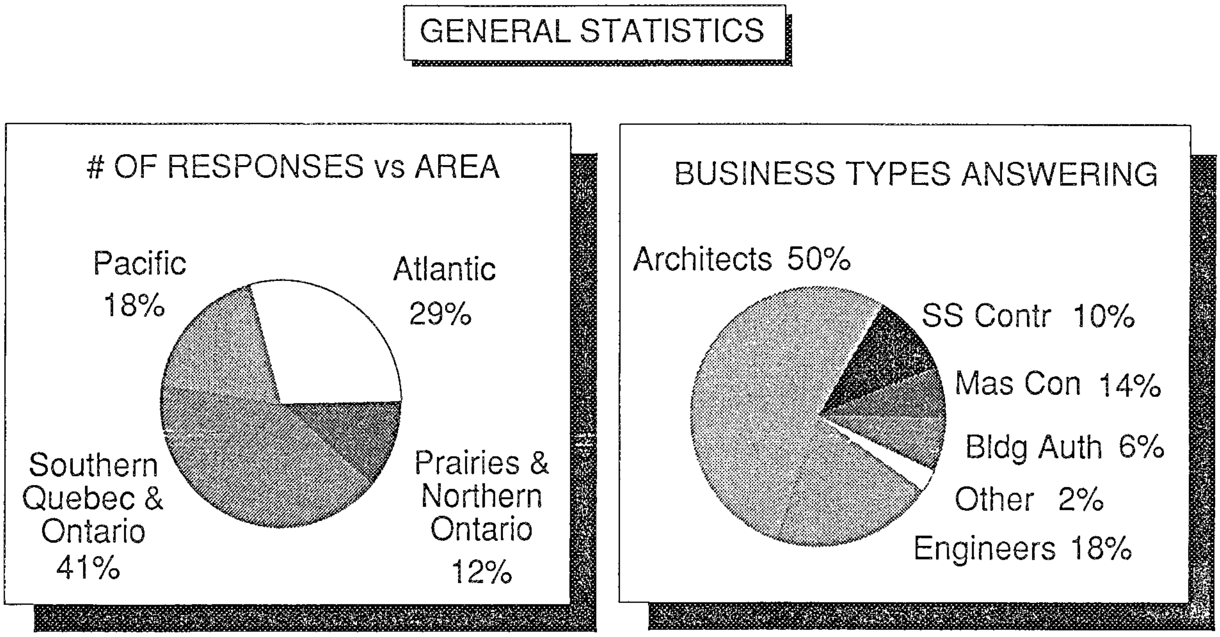



Fig. 1 General statistics on regional and business response

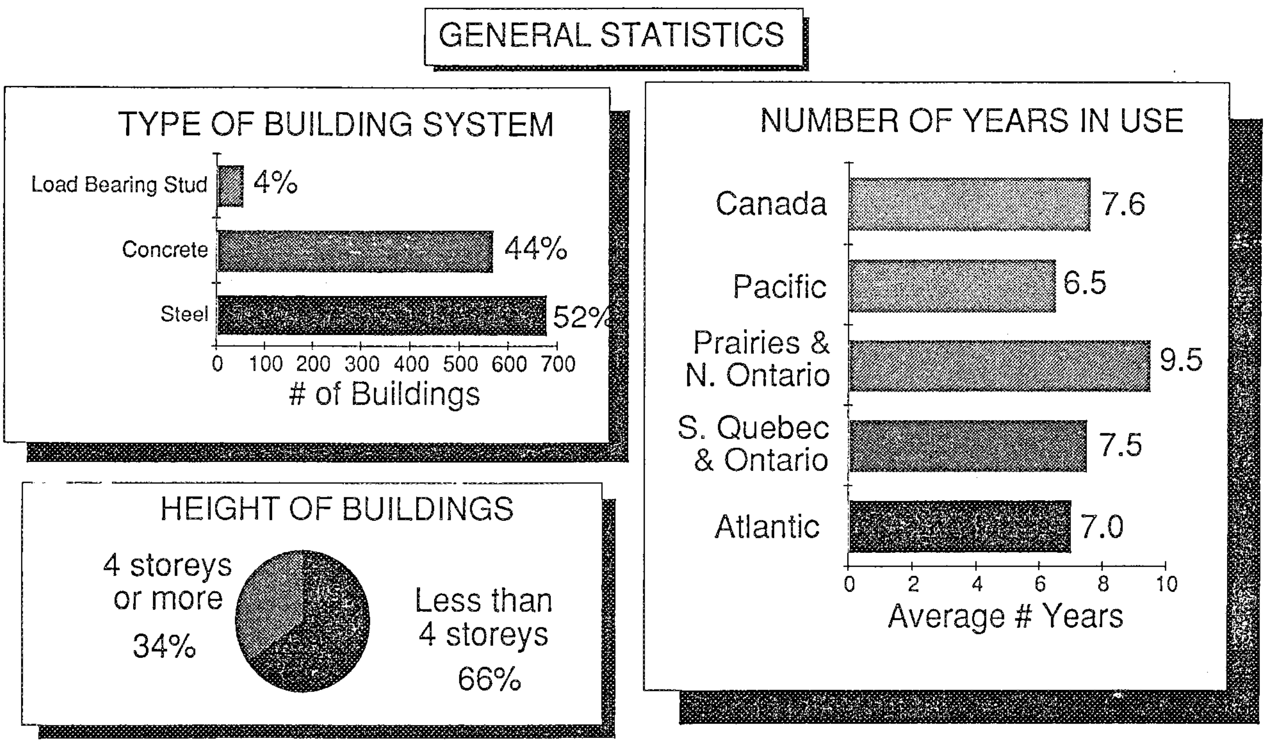



Fig. 2 General statistics building systems, building height and years of usage

moisture accumulation and therefore the potential for corrosion of metal components

- Inadequate inspection practices.

## 4.0 DESIGN ISSUES

Fig. 4 indicates that currently more than 50% of the design professionals specify a limit of L/360 for lateral steel stud deflection under wind loading. About 20% use the more stringent requirement of L/600 and about 10% use a permissible deflection limit of L/240 or greater. L represents the height of a simply supported steel stud.

The question as to which party most frequently designs the BV/SS walls was surveyed and the findings are also included in Fig. 4. It appears that the project architect most often designs the BV/SS wall system. The project structural engineer seems to assume this responsibility in less than 30% of the cases. The survey further indicates that shop drawings are not often required in the specifications for the BV/SS wall system.

The questionnaire attempted to determine what standards or guidelines are being used in the design of the BV/SS wall system. As shown in Table 2, a wide variety of standards and guidelines are being used by designers. The survey indicates that about one third of the designers rely on product catalogues and industry publications, while another third use CSA standards, NBC and local codes. About 13% design the BV/SS wall system based on common sense and intuition.

Table 2 Currently Used Standards and Guidelines

| STANDARDS OR GUIDELINES USED | PERCENTAGE |
|---|---|
| Product catalogues and industry publications | 34 |
| CSA standards, NBC codes and publications, local codes, CMHC reports and publications | 27 |
| No codes, but common sense and intuition | 13 |
| Structural engineer's opinion | 8 |
| Masonry Institute publications and literature | 7 |
| Architectural standards and specifications, foreign codes and guidelines | 6 |
| Past and current research findings | 5 |

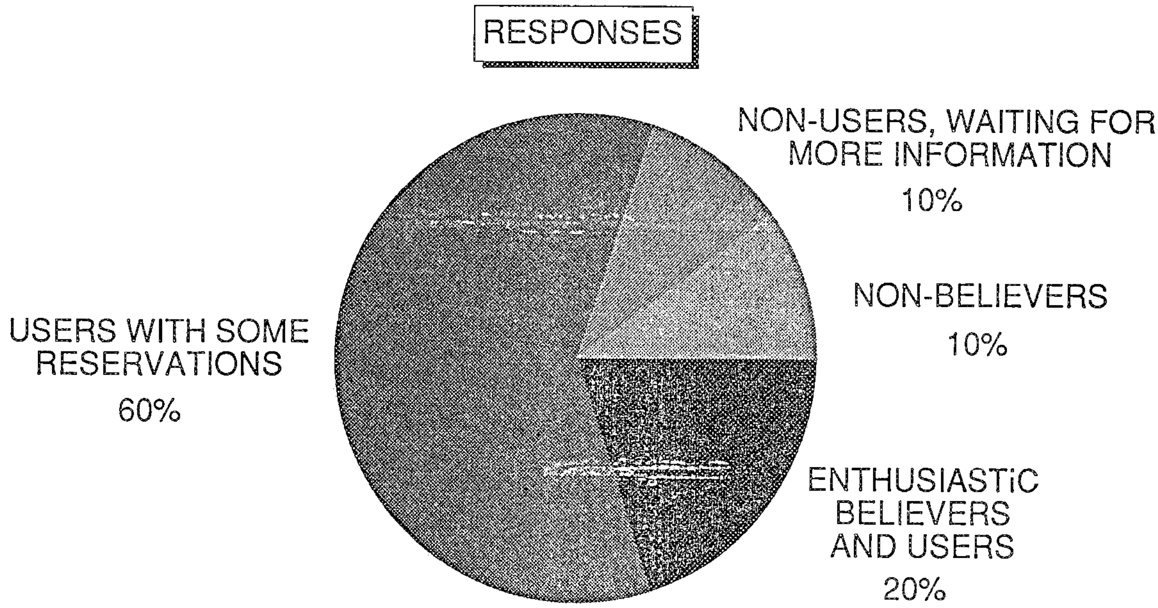



Fig. 3 Groups of responses

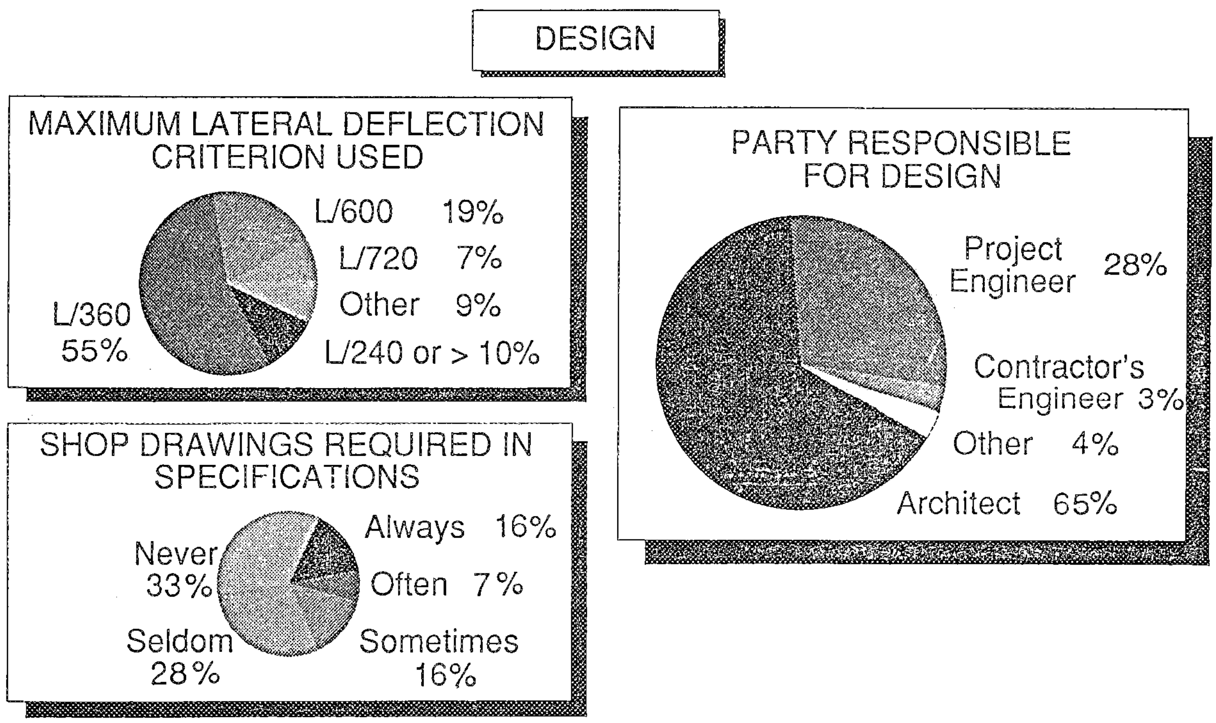



Fig. 4 Information on design related questions

## 5.0 CONSTRUCTION PRACTICES

Current construction practices are summarized in Figs. 5 to 9.

### 5.1 Stud Information (Figs. 5 and 6)

Stud size: For all applications, almost the whole spectrum of stud sizes is used, but taking the average for the various applications, 25% indicated as a first choice the 92mm, 20ga. stud, 22% the 152mm, 20ga. stud and 22% the 152mm, 18ga. stud.

Heavier studs are generally used for commercial and high rise applications, while lighter studs are chosen for low rise residential buildings.

Stud spacing: The most commonly used stud spacing is 400mm, chosen by 71% of the respondents while 19% say they use a 600mm spacing and only 8% use a 300mm spacing.

Several respondents indicated that they will generally reduce the stud spacing for storey heights greater than 2.4m and one respondent mentioned that he will double up on studs with a spacing of 400mm or even 300mm for floor heights in excess of 3.0m.

Stud corrosion protection: 97% of respondents use galvanized studs while 3% use other means. None of them use paint.

Studs around window openings: 82% specify double studs on each side of window openings, 14% use single studs and 4% use other details.

Place of fabrication: 95% of the BV/SS walls are stick-built on site.

Method of assembly of wall panels: The most commonly used fastening method is screw fastening (74%), followed by a combination of welding and screw assemblage (20%). Welding alone is used by only 6% of the respondents.

Stud/ceiling track connection: 68% permit movement, 25% use fixed connections, 7% leave it up to the contractor. A slightly higher percentage of sliding connections are used in high rise buildings.

Top and bottom track attachment: Power actuated fasteners are favored at 70% while embedded anchor bolts, self

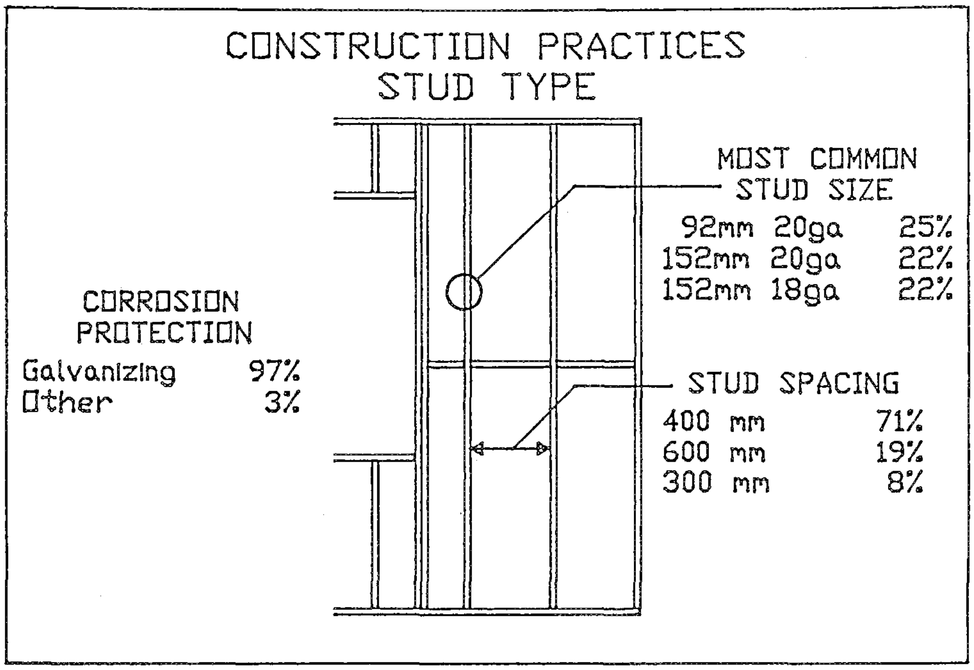



Fig. 5 Information on stud size, stud spacing and stud corrosion protection

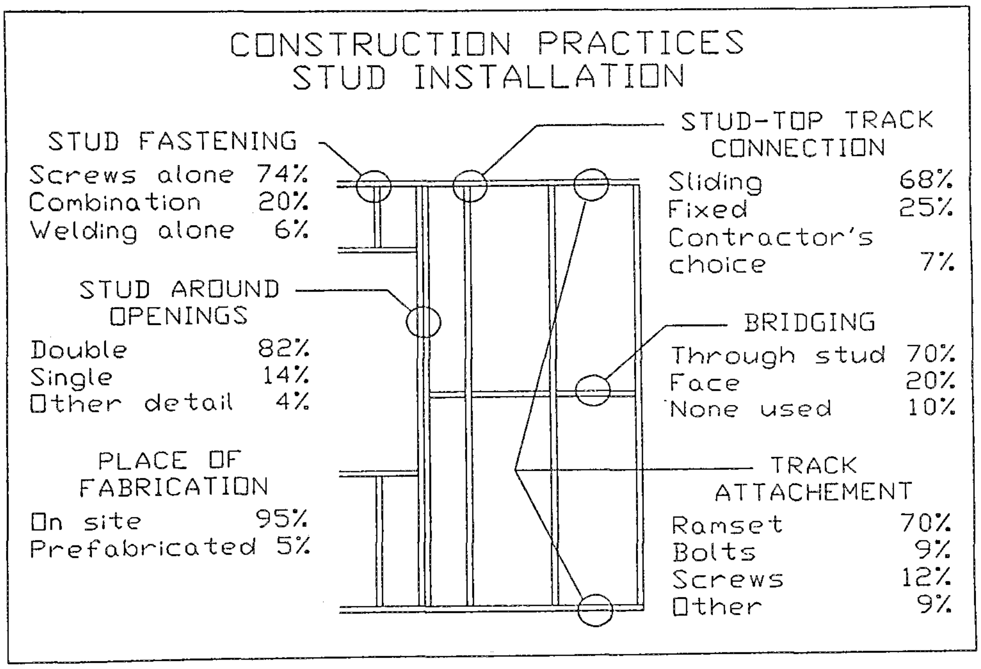



Fig. 6 Information on stud installation details

tapping screws and other devices share the remaining percentages approximately equally.

<u>Type of steel bridging</u>: Bridging through the knockout is used by 70% of the respondents, 20% use face bridging and 10% don't use bridging at all.

## 5.2 Brick Ties (Fig. 7)

Adjustable ties, screwed to the studs are used by 50% of the respondents, 28% use adjustable ties directly engaged with the studs, and 22% of the respondents use corrugated strip ties.

A good number of designers feel that strip ties should not be used as they may buckle in wider wall cavities.

Galvanizing is the most popular type (90%) of corrosion protection for brick ties.

## 5.3 Sheathing and Insulation (Fig. 8)

<u>Inner sheathing</u>: Little difference exists between low rise and high rise buildings as far as inner sheathing is concerned. 47% of the respondents indicated that they specify drywall only and about 46% use both drywall and insulation. For buildings less than four storeys, 5% will use insulation alone.

<u>Outer sheathing</u>: Again little difference exists between high and low rise buildings. 36% use drywall only, 36% use drywall and insulation and 22% use rigid or semi-rigid insulation alone.

<u>Insulation</u>: Two thirds use fiber batt insulation inside the wall cavity while one third use rigid insulation outside the stud wall.

## 5.4 Other Details (Fig. 9)

<u>Caulking at Tracks</u>: While 80% specify or apply caulking at the top and bottom tracks, it was pointed out that during cold weather installation caulking frequently hardens before the track can be installed. Under these circumstances, it is frequently difficult to ensure an air tight seal. For this reason, some contractors in the Ottawa area use a foam strip gasket and caulking at the top

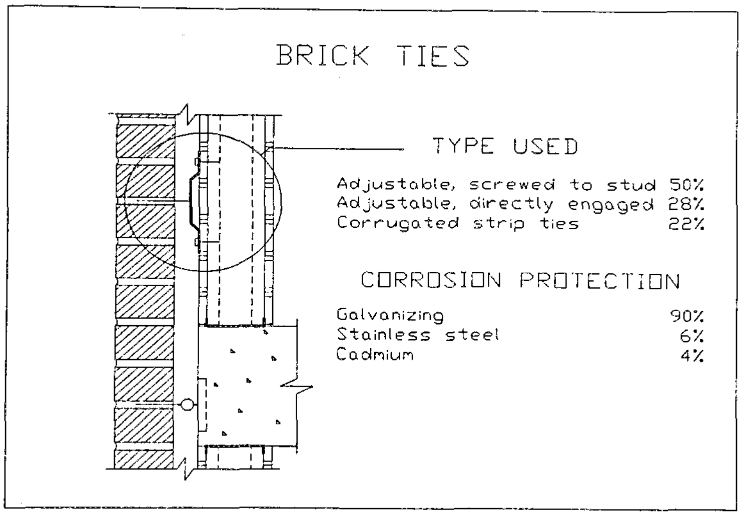



Fig. 7 Information on brick ties and corrosion protection

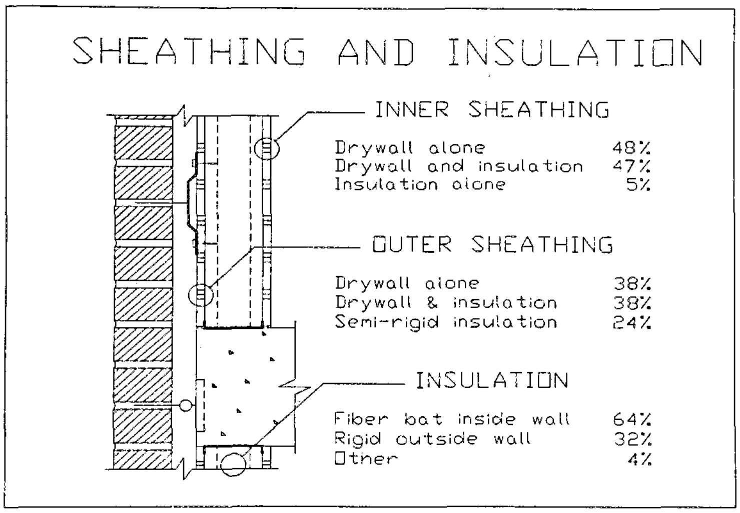



Fig. 8 Information on sheathing and insulation materials

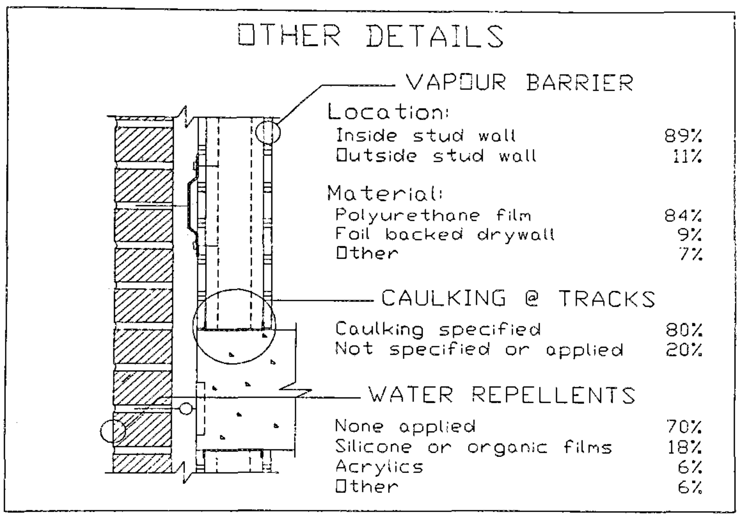



Fig. 9 Information on vapour barrier, track caulking and water repellents for masonry veneer

and bottom track/slab interface as well as perimeter caulking to ensure air tightness.

Vapour barrier: 89% of the time, the vapour barrier is specified inside the stud wall rather than outside. Polyurethane film is the most popular vapour barrier material (84%). Foil-backed drywall is used in about 9% of cases.

## 6.0 DETAILING

Contractors commented that professionals are often not willing or able to provide the necessary technical information.

This feeling is strongly supported by the survey, as illustrated in Fig. 10, where about 80% of steel stud/drywall contractors say they usually don't get a clear description of steel stud wall details with the tender drawings. They also reported frequent discrepancies between structural and architectural detail drawings for BV/SS walls.

## 7.0 BREAKDOWN OF WORK BETWEEN TRADES

The response to the question on how well the breakdown of work between trades is defined is illustrated in Fig. 11. While 70% of the steel stud/drywall contractors feel the breakdown of work between them and the masonry contractor is not very well defined, only 30% of the masonry contractors surveyed feel that way.

Since the survey indicates that steel stud/drywall and masonry contractors are most often responsible for constructing and installing the components of the BV/SS walls, future training efforts should be directed towards these two trades.

## 8.0 INSPECTION

All groups widely perceive inspection to be particularly important for this type of construction. The current level

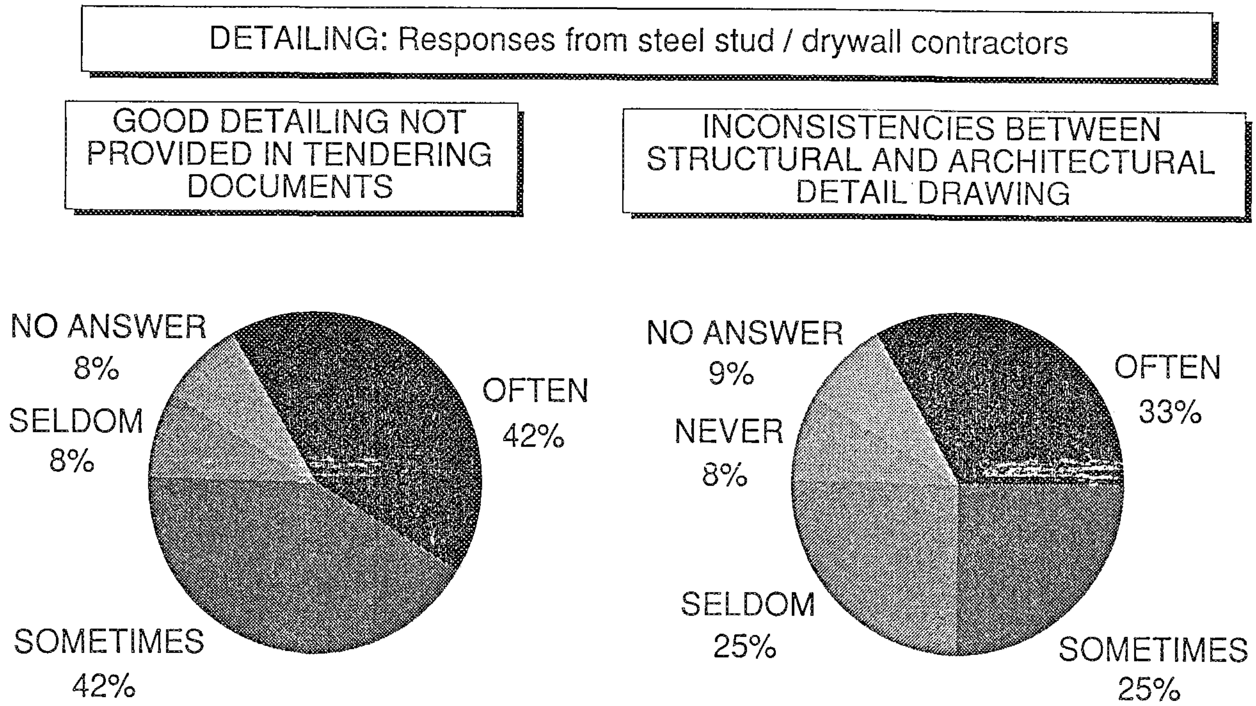



Fig. 10 Contractors responses to questions on detailing

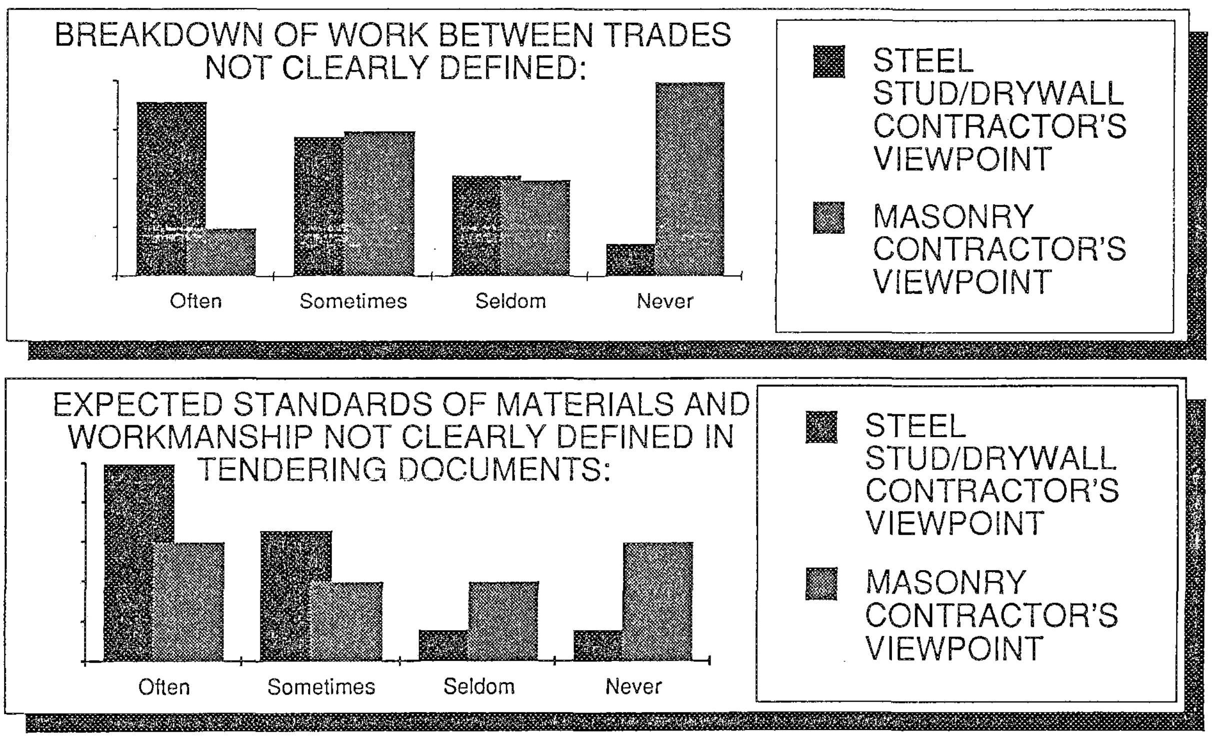



Fig. 11 Contractors responses to questions on work breakdown and expected standards of materials and workmanship

and quality of inspection is thought to be inadequate. As seen in Fig. 12, the survey showed that although inspection is usually performed, in about 50% of the cases it is performed at completion of steel stud construction, sheathing and insulation or at a later stage.

This means that the steel stud structural integrity frequently cannot be verified.

## 9.0 PROBLEMS DURING CONSTRUCTION

Professionals were asked to list the most common problems encountered with the system during construction and one third of the respondents indicated workmanship, inexperienced tradesmen or non-compliance with specifications to be a major problem. This points to the urgency for more education and technology transfer.

While few respondents have reported corrosion of metal components to be a problem at the present time, a good number have shown great concern about the long term performance of the system because of the risk of corrosion. The most common construction problems encountered by professionals are summarized in Table 3.

Table 3 Most Common Problems Encountered by Professionals with the Construction of BV/SS Walls

| PROBLEMS | PERCENTAGE |
|---|---|
| Poor workmanship, inexperienced tradesmen, wrong trade, non-compliance with specifications............... | 31 |
| Poor waterproofing and corrosion protection, discontinuity of air/vapour barrier and insulation...... | 20 |
| Inadequate brick ties or brick tie connections.......... | 16 |
| Inadequate stud thickness and framing for openings (services and windows), inadequate bridging............. | 9 |
| Improper stud connections and anchoring................. | 7 |
| Excessive stud deflection due to wind load.............. | 2 |
| Other (moisture penetration, cold bridging, poor stud assembly)......................................... | 15 |

## 10.0 INFORMATION AND RESEARCH NEEDS

Upon reviewing the comments in each questionnaire, it emerges that most respondents, regardless of their group, feel that the level of knowledge and understanding or expertise in the field of brick veneer/steel stud construction is inadequate. A very high percentage have expressed the need for standardization, and some consider this to be an urgent requirement. The responses to these questions are illustrated in Fig. 13.

All groups seem to perceive the BV/SS wall system as sensitive both to design and construction. Compared to the masonry backup system, it is thought of as being less forgiving to error at either level of implementation.

Professionals have also often expressed their own need for more information on the behaviour and strength of the BV/SS wall system, frequently pointing at particular areas they felt they should know more about. Items most frequently mentioned are:

- brick ties - type
  - strength
  - spacing
- longterm performance of the wall system
- corrosion of metal components
- maximum deflection criteria
- detailing - stud/track connection
  - bridging

In general, it appears that professionals are seeking improvements in the level of technological knowledge as well as the transfer of such knowledge to contractors.

## 11.0 GENERAL COMMENTS

The questionnaire also solicited general comments and opinions from respondents on the BV/SS wall system.

The key comments received from professionals address problems in the field and the question of professional fees. Pertaining to problems in the field, professionals mentioned primarily workmanship, inexperienced tradesmen and non-compliance with specifications as the key problems. But professionals also pointed out that fees do not usually include design and inspection of the BV/SS wall system.

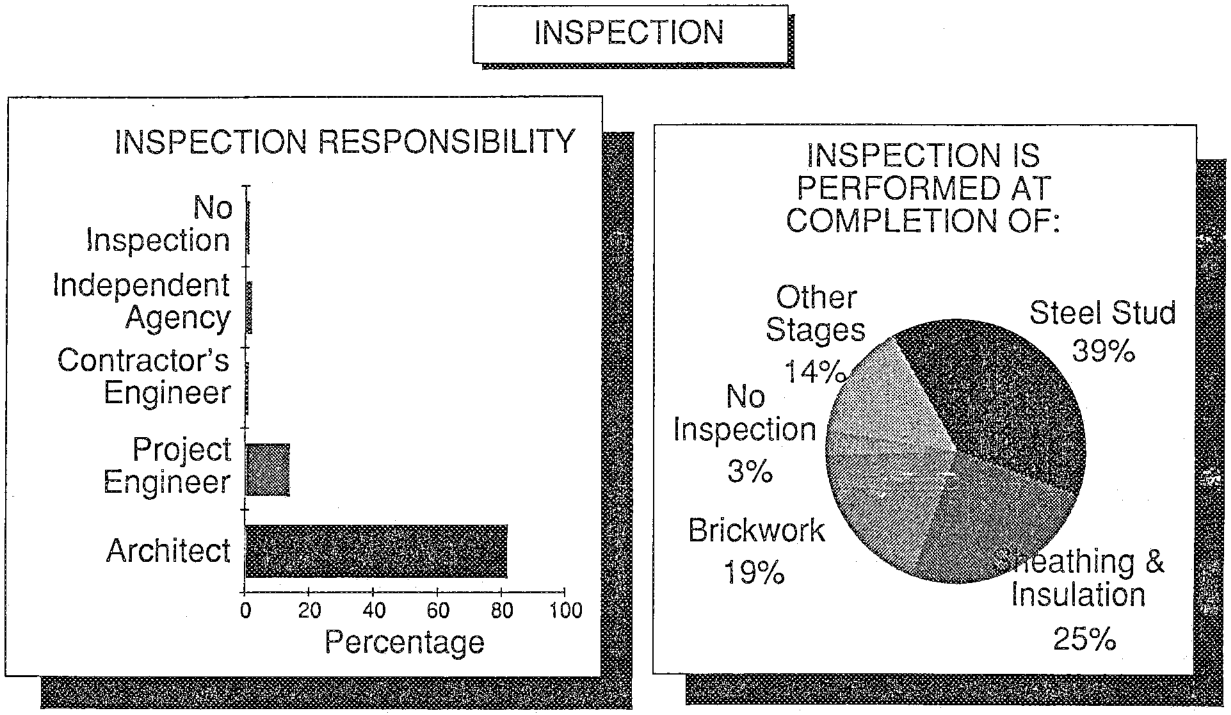



Fig. 12 Inspection requirements and responsibility

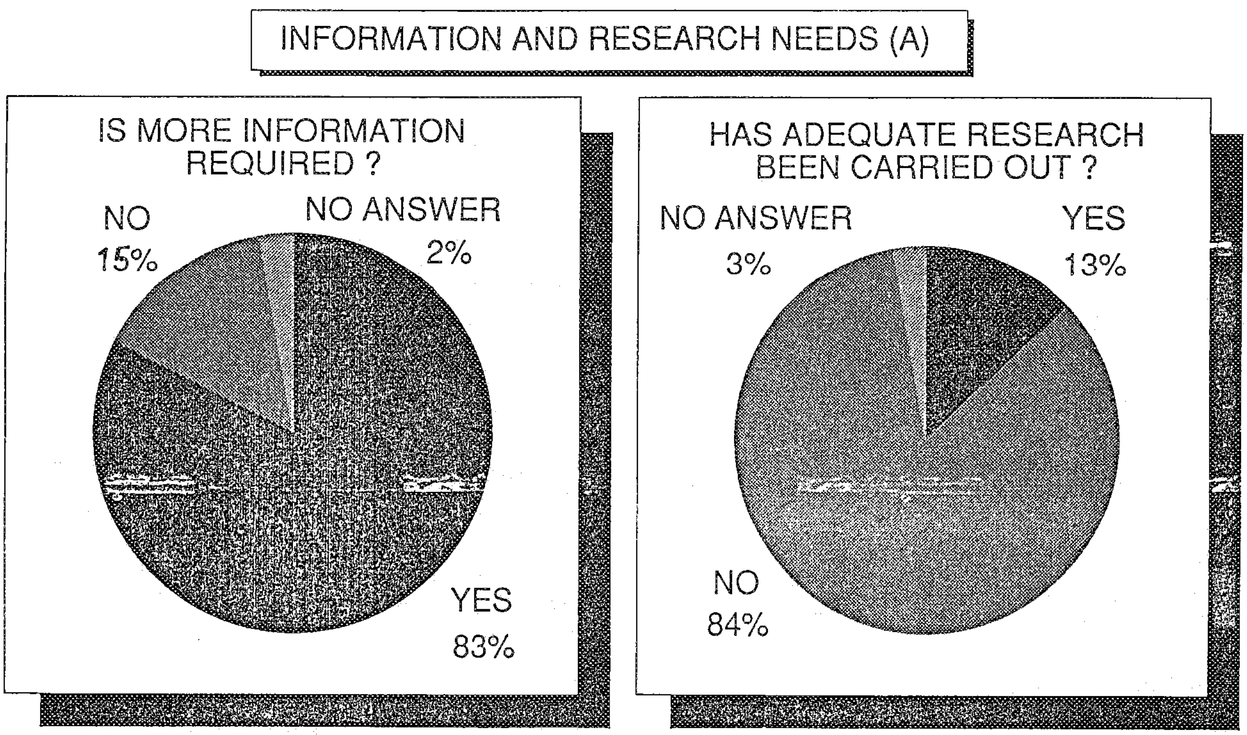



Fig. 13 Information and research requirements

The following comments were received from contractors:

- The successful performance of the BV/SS wall system depends on proper installation and inspection.
- Details are frequently not clearly defined.
- Generally, better professional support is required. Professionals are often not willing to provide required technical information.
- Architects and Engineers often refuse to accept responsibility for the design of the BV/SS wall system, leaving it up to contractors to choose materials and member sizes.

## 12.0 SUMMARY

The main purpose of this survey was to identify and establish the state of industry concern for the BV/SS wall system in Canada. The small number of responses received in some regions of the country and the location of the respondents in these regions make the case of regional analysis an impractical task. However, the findings from the four surveys can be summarized under the following six headings.

### 12.1 Uses of the BV/SS Wall System Across the Country

The BV/SS wall system is used across Canada both in residential and commercial buildings. The buildings' structural system typically, consists of steel framing, concrete framing or loadbearing steel stud walls. The BV/SS wall system is more popular in buildings less than four storeys tall. The number of years that this system has been in use varies across the country from a low of 6.5 years in Pacific Canada to a high of 9.5 years in the Prairie Region and Northern Ontario. The Nation's average use of the BV/SS wall system is approximately 8 years.

### 12.2 BV/SS Wall Design

The project architect is most often the designer and inspector of BV/SS walls during construction. No standardized procedures for inspection during construction exist. The design is based on product catalogues and related industry publications.

Regarding stud spacing, the 400 mm o.c. steel stud spacing is the most popular choice. In buildings less than 4 storeys tall, the 92 mm C-stud is most frequently used. In taller buildings, the 152 mm C-stud is used most often. The use of the double stud as a vertical trimmer around window openings is common practice. Galvanizing was found to be the most popular type of corrosion protection for steel studs and brick ties.

Inner sheathing materials most often consist of either drywall only or insulation and drywall. Outer sheathing materials are most often made up of drywall and insulation. Fibre batts within the stud space are the most common insulation material in use. The application of caulking between building slabs and top and/or bottom tracks is common practice.

The most popular vapour barrier consists of plastic film (poly) and it is generally applied on the inside face of the steel studs. Adjustable ties, screwed directly to the studs are the most popular means of securing the brick veneer. The maximum lateral stud deflection limit is specified most frequently as L/360.

## 12.3 BV/SS Wall Construction

BV/SS walls are most often stick-built on the job site using screw fasteners. A connection permitting slab movement is commonly used between studs and ceiling tracks. The use of power activated fasteners to attach bottom and top tracks is common practice and the bracing of steel studs is most often done through the steel stud knockouts. Since the steel stud/drywall contractors and the masonry contractors are most often involved in the installation of the BV/SS wall system, future education and technology transfer efforts should be directed primarily towards these two trades.

## 12.4 In-Service Problems

The majority of respondents had encountered BV/SS walls which were not performing satisfactorily. The most common deficiency appears to be air infiltration. Improved detailing and installation practices on air barriers are required. Corrosion of metal components such as studs, screws and ties as well as excessive stud flexibility have not generally been observed as major problems.

### 12.5 Additional Research and Technology Transfer Requirements

The need for more information and research is recognized by all respondents. The most common problems related to design and construction of BV/SS walls could be reduced with a better transfer of existing and future technology, and with additional training.

### 12.6 Recommendations

The key recommendations offered by the respondents are as follows:

- conduct more research on:
  - longterm performance of the BV/SS wall system
  - deflection/stiffness characteristics
  - strength of ties
- prohibit the use of strip ties and screwed-on ties
- produce good standards and design aids
- improve level of inspection
- encourage that design is performed by professionals who are competent on the subject.

### 12.7 Conclusions

The survey showed clearly that the system has its supporters but that there is considerable concern over it's safety in the long term. A good number say they would not use the system at this time.

Both builders and consultants widely expressed their need for more information on the topic. Contractors stressed their need for better professional support while architects and engineers expressed their need for more technical information.

The great majority would like more research to be conducted on the BV/SS wall system. It appears that the publication of an official standard would be welcomed by many respondents.

# CHAPTER 2 FIELD SURVEY

## 1. INTRODUCTION

A total of 8 buildings in 4 cities have been investigated to determine construction practices across Canada and in-service performance of the BV/SS wall system under various climatic exposure conditions. Two buildings each were inspected in the following locations:

- St. John's, Newfoundland
- Montreal, Quebec
- Toronto, Ontario
- Calgary, Alberta

The investigation consisted firstly of a visual examination of the brick veneer, secondly of a detailed inspection of the exterior wall system in a number of locations through inspection openings from the interior and thirdly of a thermographic survey.

## 2. THE BUILDINGS

The buildings to be investigated were selected by CMHC through their field office personnel and the particulars about each building are summarized in Table 1.

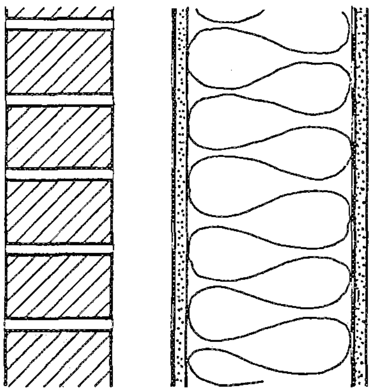
Table 1 Building Particulars

| Building No. | Location | Age (Yrs) | No. of Storeys | Structural System |
|---|---|---|---|---|
| 1 | St. John's | 8 | 7 | Prestressed Concrete |
| 2 | St. John's | 6 | 6 | Slabs, Steel Columns |
| 3 | Montreal | 9 | 8 | Steel Frame/R.C. Slabs |
| 4 | Montreal | 4 | 6 | Reinforced Concrete |
| 5 | Toronto | 8 | 9 | Reinforced Concrete |
| 6 | Toronto | 11 | 7 | Reinforced Concrete |
| 7 | Calgary | 9 | 11 | Prestressed Concrete |
| 8 | Calgary | 9 | 17 | Slabs, Concrete Columns |

## 3. EXTERIOR WALL DETAILS

The typical as-built exterior wall details for each building are outlined below.

### B U I L D I N G # 1 ( ST. JOHN'S )



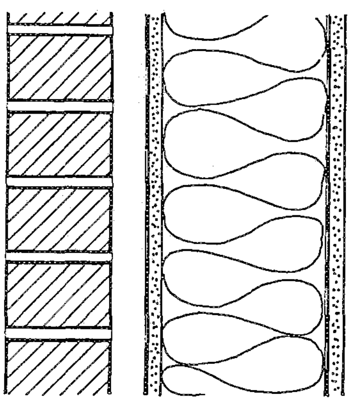
**WALL SECTION**

- 100mm BRICK MASONRY
- 55mm AIR SPACE
- BUILDING PAPER
- 13mm DRYWALL
  BATT INSUL. IN STUD SPACE
- 152mm STEEL STUD
- 13mm DRYWALL
  FOIL-BACKED

**STUD WALL INFORMATION**

- 152mm STUD
- 609mm SPACING
- SCREWED TO TOP AND BOTTOM TRACK ON INSIDE
  NO BRIDGING

### B U I L D I N G # 2 ( ST. JOHN'S )



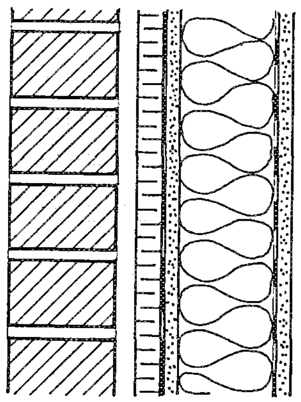
**WALL SECTION**

- 100mm BRICK MASONRY
- 30mm AIR SPACE
- BUILDING PAPER
  13mm DRYWALL
- BATT INSUL. IN STUD SPACE
- 152mm STEEL STUD
- POLY. VAPOUR BARRIER
- 16mm DRYWALL

**STUD WALL INFORMATION**

- 152mm STUD
- 406mm SPACING
- SCREWED TO TOP AND BOTTOM TRACK ON INSIDE
- THROUGH-THE-
- STUD BRIDGING

## B U I L D I N G # 3 ( MONTREAL )



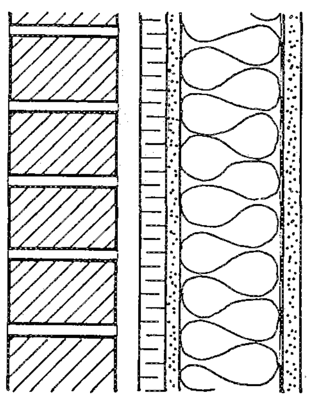
WALL SECTION

- 100mm BRICK MASONRY
- 18mm AIR SPACE
- 25mm RIGID INSULATION
- BUILDING PAPER
- 13mm EXTERIOR DRYWALL
- BATT INSUL. IN STUD SPACE
- 92mm STEEL STUD
- POLY. VAPOUR BARRIER
- 13mm DRYWALL

STUD WALL INFORMATION

- 92mm STUD
- 406mm SPACING
- SCREWED TO TOP AND BOTTOM TRACK ON INSIDE
- NO BRIDGING

## B U I L D I N G # 4 ( MONTREAL )



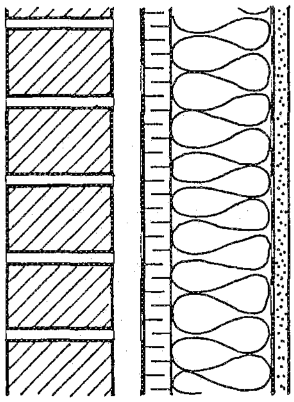
WALL SECTION

- 100mm BRICK MASONRY
- 20mm AIR SPACE
- 25mm SEMI-RIGID INSULATION
- 13mm DRYWALL
- BATT INSUL. IN STUD SPACE
- 92mm STEEL STUD
- POLY. VAPOUR BARRIER
- 16mm DRYWALL

STUD WALL INFORMATION

- 92mm STUD
- 406mm SPACING
- SCREWED TO TOP AND BOTTOM TRACK ON INSIDE
- NO BRIDGING

## B U I L D I N G # 5 ( TORONTO )



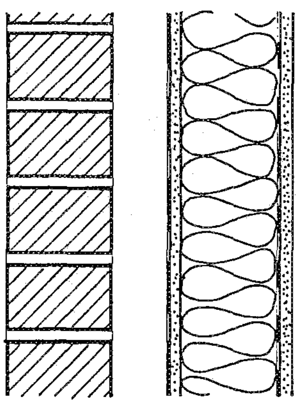
WALL SECTION

- 100mm BRICK MASONRY
- 25mm AIR SPACE
- BUILDING PAPER
- 25mm RIGID INSULATION
- BATT INSUL. IN STUD SPACE
- 92mm STEEL STUD
- POLY. VAPOUR BARRIER
- 13mm DRYWALL

STUD WALL INFORMATION

- 92mm STUD
- 406mm SPACING
- SCREWED TO TOP AND BOTTOM TRACK ON INSIDE
- FACE BRIDGING

## B U I L D I N G # 6 ( TORONTO )



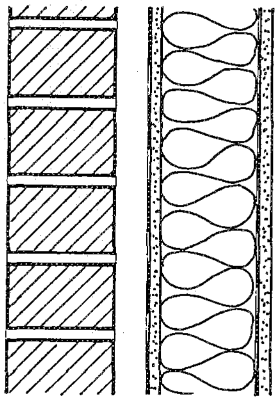
WALL SECTION

- 100mm BRICK MASONRY
- 50mm AIR SPACE
- BUILDING PAPER
- 11mm TENTEST
- BATT INSUL. IN STUD SPACE
- 89mm STEEL STUD
- POLY. VAPOUR BARRIER
- 13mm DRYWALL

STUD WALL INFORMATION

- 89mm INTERIOR STUD
- 406mm SPACING
- NOT SCREWED TO TOP AND BOT TRACK
- NO BRIDGING

## B U I L D I N G # 7 ( CALGARY )



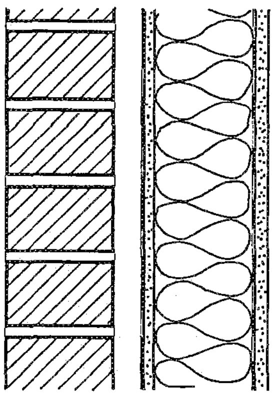
WALL SECTION

- 100mm BRICK MASONRY
- 30mm AIR SPACE
- BUILDING PAPER
- 11mm DRYWALL
- BATT INSUL. IN STUD SPACE
- 89mm STEEL STUD
- POLY. VAPOUR BARRIER
- 13mm DRYWALL

STUD WALL INFORMATION

- 89mm INTERIOR STUD
- 406mm SPACING
- SCREWED TO TOP AND BOTTOM TRACK ON INSIDE
- NO BRIDGING

## B U I L D I N G # 8 ( CALGARY )



WALL SECTION

- 100mm BRICK MASONRY
- 25mm AIR SPACE
- BUILDING PAPER
- 11mm DRYWALL
- BATT INSUL. IN STUD SPACE
- 89mm STEEL STUD
- POLY. VAPOUR BARRIER
- 13mm DRYWALL

STUD WALL INFORMATION

- 89mm INTERIOR STUD
- 406mm SPACING
- SCREWED TO BOT TRACK ONLY ON INSIDE
- NO BRIDGING

## 4. CONDITION OF BRICK VENEER

The veneer of each building was carefully scanned with binoculars from the ground, and randomly inspected from balconies and through windows at re-entrant building corners.

Deficiencies such as cracking, spalling and efflorescence staining were recorded. The condition of the brick veneer ranged from satisfactory to poor in some buildings.

Deficiencies typically included:

- spalling at shelf angle locations
- spalling due to freeze/thaw action
- cracking at corners
- cracking between openings
- cracking due to poor detailing and construction practices such as discontinuous shelf angles and rowlock sills or copings.

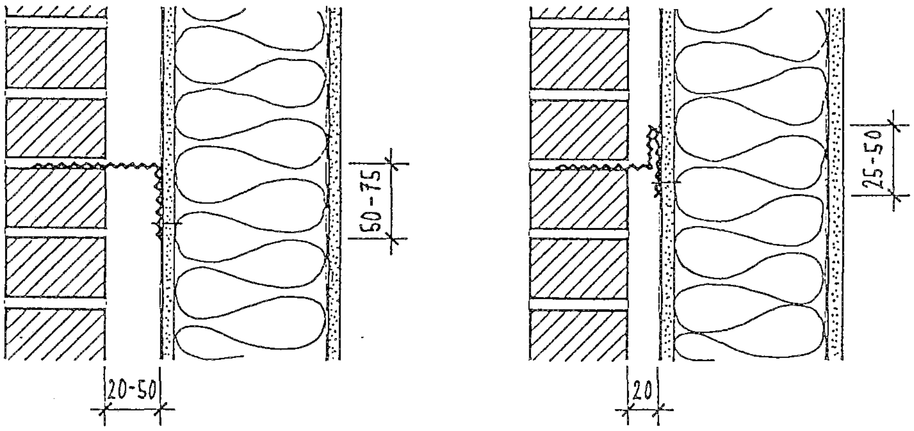
The types of deficiencies observed were found to be common to other backup wall systems and no correlation could be drawn to the use of the steel stud backup system.

## 5. INTERIOR INSPECTION

The information typically recorded during the interior inspections included:

- type and condition of brick ties
- size, spacing and anchorage of studs
- size and anchorage of top and bottom tracks
- type of bridging
- vertical movement capability of studs
- type and location of insulation
- type of moisture barrier
- type of vapour barrier
- condition of drywall
- size and condition of cavity
- brick veneer workmanship.

Over 60% of the buildings investigated utilize strip ties to fasten the brick veneer to the steel stud wall. The remainder used adjustable type ties directly screwed onto the stud flange. The strip tie installation was generally poor with ties being frequently bent up or down to suit the installation as shown typically below:




The condition of the brick ties varied from building to building. Strip ties generally showed corrosion at the brick/cavity interface or where in contact with mortar, i.e. if covered with mortar droppings. Adjustable ties were generally in satisfactory condition but corrosion was also evident in some instances especially at the brick/mortar interface.

Stud sizes varied from 150 mm 20 ga. to 89 mm 26 ga. interior type stud with spacings of mostly 400 mm as summarized in Table 2.

Table 2 Stud Sizes and Spacings

| Building No. | Location | Size | Thickness Without Coating (mm) | Gauge | Spacing |
|---|---|---|---|---|---|
| 1 | St. John's | 150 | 0.876 | 21 | 609 |
| 2 | St. John's | 150 | 0.965 | 20 | 406 |
| 3 | Montreal | 92 | 0.914 | 20 | 406 |
| 4 | Montreal | 92 | 0.907 | 22 | 406 |
| 5 | Toronto | 92 | 1.130 | 19 | 406 |
| 6 | Toronto | 89 | 0.478 | 26 | 406 |
| 7 | Calgary | 89 | 0.457 | 26 | 406 |
| 8 | Calgary | 89 | 0.445 | 26 | 406 |

Top and bottom tracks were generally of the same size as the studs with flanges of equal size. Table 3 gives a summary of the top and bottom tracks found in the investigation. In all cases top and bottom tracks were attached to the structure by ramset fasteners at 600 to 900 mm spacing.

Table 3 Top and Bottom Tracks

| Building No. | Top and Bottom Track | Track Thickness |
|---|---|---|
| 1 | 30 150 | 19 ga. (1.080 mm) |
| 2 | 30 150 | 19 ga. (1.080 mm) |
| 3 | 32 150 | 20 ga. (0.902 mm) |
| 4 | 25 92 * | 20 ga. (0.978 mm) |
| 5 | 25 92 | -- -- |
| 6 | 26 89 | -- -- |
| 7 | 29 89 | 26 ga. (0.445 mm) |
| 8 | 29 89 | 27 ga. (0.416 mm) |

* Top track flange = 44 mm

Bridging or bracing between studs has not been used in general. One building utilized through-the-stud channel bridging but a positive connection had been omitted. Another building used 60 mm wide, 1.17 mm (18 ga.) galvanized strapping on the exterior face of studs at mid-height of the wall.

Vertical movement capability was not provided in any of the buildings investigated, although one building had neither top nor bottom connections and another building had only bottom connections. In all cases top and bottom connections consisted of one drywall screw attached at the inner stud flange.

The presence of double studs at window openings was also reviewed. The findings are summarized in Table 4.

Table 4  Window Stud Framing Details

| Building No. | Vertical Studs at Windows |
|---|---|
| 1 | single stud |
| 2 | double stud |
| 3 | single stud |
| 4 | double stud |
| 5 | double stud |
| 6 | single stud |
| 7 | single stud |
| 8 | single stud |

The findings indicate that more than half of the buildings investigated use single studs at wall openings.

Pertaining to insulation, all buildings had glass fibre batt insulation within the stud space. The buildings in Montreal in addition had 25 mm rigid or semi-rigid insulation on the exterior together with 13 mm exterior drywall. One Toronto building also used an additional 25 mm rigid insulation on the exterior of the studs but without exterior drywall.

The vapour barrier in all but one case consisted of polyethylene sheeting placed on the warm side of the insulation. One building used foil-backed drywall. The moisture barrier consisted of building paper. One building did not have a moisture barrier. Table 5 summarizes the information on vapour and moisture barriers used.

The condition of the drywall was reviewed for the presence of moisture, mould or evidence of moisture staining. Our review indicates that in one building all wall components from the brick through to the interior drywall were wet or moist. Since the drywall on the weather side of the building exhibited surface mould, it is concluded that wet conditions occur frequently. In another building moisture was found on the exterior drywall, but in general drywall was found to be dry at the time of the investigation.

Table 5 Moisture and Vapour Barrier

| Building No. | Moisture Barrier | Vapour Barrier |
|---|---|---|
| 1 | building paper | foil-backed drywall |
| 2 | building paper | polyethylene ( - ) |
| 3 | building paper | polyethylene ( 6 mil) |
| 4 | none | polyethylene (10 mil) |
| 5 | building paper | polyethylene (6 mil) |
| 6 | building paper | polyethylene (4 mil) |
| 7 | building paper | polyethylene (2 mil) |
| 8 | building paper | polyethylene (2 mil) |

The cavity size varied from a low of 6 mm to a maximum of 64 mm. In general the cavity size was considered to be inadequate to permit proper venting, especially in view of the frequent presence of heavy mortar droppings in the cavity. Table 6 gives an overview of the recorded cavity size and condition of the cavity space.

Table 6 Cavity Size and Condition

| Building No. | Cavity Size (mm) | Condition |
|---|---|---|
| 1 | 50 - 64 | extensive mortar droppings |
| 2 | 19 - 38 | moderate mortar droppings |
| 3 | 6 - 32 | moderate mortar droppings |
| 4 | 13 - 32 | extensive mortar droppings |
| 5 | 20 - 30 | extensive mortar droppings |
| 6 | 45 - 55 | moderate mortar droppings |
| 7 | 13 - 50 | extensive mortar droppings |
| 8 | 20 - 30 | extensive mortar droppings |

Masonry workmanship throughout was considered average for residential construction. Poorly filled head joints, excessive mortar protrusions into the cavity and extensive mortar droppings on brick ties and at the flashing levels were frequently encountered.

Shadowing of studs on the interior was observed in 5 buildings. This condition occurred primarily in buildings which did not use additional insulation on the exterior face of the stud, except for one case where shadowing also occurred in spite of the presence of exterior insulation but where no exterior drywall had been used. The two buildings which used both exterior drywall and exterior insulation did not exhibit shadowing of steel studs on the interior finishes.

Moisture conditions within the exterior wall assembly varied from location to location depending on prevailing weather conditions. Relative humidity in the cavity was generally above 50% with values as high as 82%. Where rain had occurred just a few days prior to the inspection, the interior face of the brick veneer and brick ties were found to be wet.

## 6. METAL COMPONENTS

Metal components such as studs, tracks, and brick ties were examined in detail to determine material thickness, corrosion protection coatings and extent of corrosion. Based on this metallurgical assessment remaining life predictions were made. Table 7 provides a summary of the findings.

As seen from Table 7, there is a wide contrast in the condition of the metal specimens. While many of them show no degradation, some of them are extensively corroded. Electrolytically applied zinc coatings appear to show better resistance to corrosion than the hot dipped systems, in spite of a thinner coating. Hot dipped zinc coatings range between 0.001 to 0.002 in. (0.025-0.051 mm). If the amount of zinc coating is compared to service life it appears that the coating thickness on the metal components tested are at the low end of the scale. Fig. 1 shows that for a coating thickness of 0.001 in. (0.025 mm) the underlying steel would be protected for about 15 years in a rural environment whereas an increase in coating thickness to 0.002 in. (0.051 mm) or 0.003 in (0.076 mm) would increase the service life to say 30 years or 40 years, respectively. On the other hand, if the system is exposed to a more aggressive environment, a coating of 0.001 in. (0.025 mm) would protect the underlying steel for only a few years and a heavier coating would still only provide about 5 to 8 years of protection. This indicates the absolute need to stop the ingress of moisture or to improve the protection of vulnerable steel components.

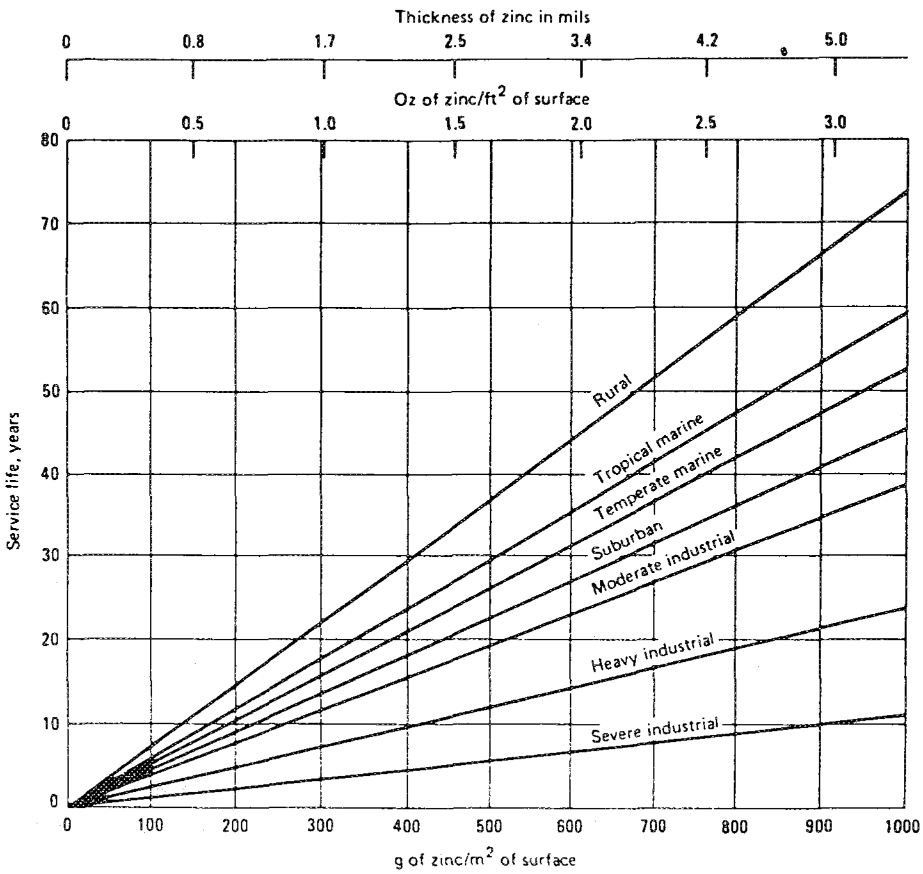
Table 7  Metallurgical Examination of Metal Components

| Building No. | Age (Yrs) | Metal Component | Thickness (mm) | Thickness Gauge (MSG) | Coating Type | Coating Thickness (mm) | Estimated Remaining Life (Yrs) |
|---|---|---|---|---|---|---|---|
| 1 | 8 | stud | 0.876 | 21 | HD | 0.033 | <5 |
| | | strip tie | 0.635 | 24 | HD | 0.036 | <2-5 |
| 2 | 6 | stud | 0.965 | 20 | HD | 0.031 | >10 |
| 3 | 9 | stud | 0.914 | 20 | HD | 0.028 | >10 |
| | | strip tie | 0.305 | 30 | HD | 0.028 | >5 |
| 4 | 4 | stud | 0.907 | 21 | HD | 0.041 | >10 |
| | | strip tie | 0.914 | 20 | HD | 0.031 | <5 |
| 5 | 8 | stud | 1.130 | 19 | HD | 0.020 | >10 |
| | | adj. tie | 4.750 | -- | EG | 0.013 | >5 |
| 6 | 11 | stud | 0.495 | 26 | EG | 0.010 | >10 |
| | | adj. tie | 0.775 | -- | EG | 0.013 | >5 |
| 7 | 9 | stud | 0.445 | 26 | EG | 0.010 | >10 |
| | | strip tie | 0.330 | 30 | HD | 0.031 | <2 |
| 8 | 9 | stud | 0.445 | 26 | EG | 0.010 | >10 |
| | | strip tie | 0.356 | 29 | HD | 0.036 | <2 |

HD = Hot dipped
EG = Electro galvanized

Remaining Life:
>10 means excellent
>5 limited life say up to 15 years
<2 very poor condition

**Fig. 1 Effect of amount of zinc on service life of galvanized sheet in various environments**




Service life is measure in years to the appearance of first significant rusting.

## 7. MORTAR CONDITIONS

Mortar samples were removed from several buildings and analysed for chloride contents. Table 8 shows a summary of percent chloride by weight of cement for the samples analysed.

Table 8 Mortar Sample Chloride Analysis

| Building No. | Sample No. | % Chloride by Weight of Cement |
|---|---|---|
| 3 | 7 | 1.10* |
| | 8 | 0.58* |
| 4 | 5 | 0.05 |
| | 6 | 0.14 |
| 5 | 1 | 0.77* |
| | 2 | 0.05 |
| 6 | 3 | 0.03 |
| | 4 | 0.04 |
| 7 | 9 | 0.26* |
| | 10 | 0.29* |
| 8 | 11 | 1.63* |
| | 12 | 0.02 |

* Exceeds ACI published threshold limit for acid-soluble chloride of 0.20% above which corrosion of steel occurs

The test results indicate that in 6 out of 8 buildings the chloride content in the mortar exceeds the ACI published threshold level of 0.20% by weight of cement above which corrosion of steel occurs. In four cases the threshold limit was exceeded by a factor of 4 to 8. It is assumed that the chlorides were added as accelerators during construction. Our findings therefore indicate that this practice is still widely used.

## 8. THERMOGRAPHIC SURVEY

Thermographic surveys were conducted on buildings in St. John's, Nfld., Toronto, Ont., and Calgary, Alta.

In all cases the thermographic surveys indicated that the building envelope was uniformly insulated and free of any significant air leakage or moisture retention at the time of the survey. Air movements across the building envelope were considered normal. The majority of the detected air leakage was associated with window/door framing details. The thermographic survey also indicated various degrees of thermal bridging at floor levels in all buildings. This condition occurred whether slab edge insulation was provided or not.

## 9. SUMMARY OF FINDINGS

The findings of the field survey of 8 buildings in 4 cities across Canada are summarized as follows:

1. Steel Stud System

- steel studs are generally in satisfactory condition and exhibiting little evidence of corrosion after a service of of 4 to 11 years
- steel stud construction is frequently substandard in the following ways:
  - lack of bridging
  - inadequate stud attachment
  - lack of vertical movement capability
  - use of thin gauge material
  - inadequate corrosion protection
  - use of black metal screws
- where corrosion has been noticed, bottom tracks were the first ones to be affected
- the steel stud assembly is not generally protected effectively against exposure to moisture.

2. Brick Veneer

- brick veneer conditions are generally satisfactory
- where brick veneer distress has been observed, the type of distress is typical also with other backup wall systems
- mortar frequently contains excessive amounts of chlorides
- brick ties are often corroded and where strip ties have been used their service life appears to be significantly shortened
- cavity space is typically too small and partially filled with mortar droppings

3. Building Science Issues

- air leakage through the wall system appears to be minimal except for isolated locations

- thermal bridging is evident at floor levels in most buildings

- shadowing of studs on interior surfaces occurs mostly where no exterior insulation has been used.

## 10 CONCLUSIONS

The following tentative conclusions are drawn:

- keep moisture out of wall assembly

- increase minimum zinc coating thickness to at least 0.002 in. or 0.003 in. (0.051 or 0.076 mm)

- avoid using black metal drywall screws

- do not use strip ties

- do not use chloride additive in mortar

## PART 2 ADVISORY DOCUMENT

**R. G. Drysdale and G. T. Suter**

### PREFACE AND ACKNOWLEDGEMENTS

In 1981 Canada Mortgage and Housing Corporation published a timely Advisory Document on "Exterior Wall Construction in High-Rise Buildings" authored by W.G. Plewes. This publication for the first time in Canada focused attention to a considerable depth on what might be termed modern masonry cavity walls and veneers on frame buildings. While such exterior wall constructions had come into wide use, a growing number of performance problems necessitated a comprehensive look at design, construction, supervision and inspection issues which potentially underlie such problems.

This new publication serves partly as an update of the 1981 document but is mainly a vehicle to discuss best design and construction practices for both the long established brick veneer/concrete masonry (BV/CM) and the relatively new brick veneer/steel stud (BV/SS) wall systems. Especially over the past 10 years, BV/SS walls have become more widely used in Canada as an economical building enclosure system. However, based on a CMHC sponsored field survey ("Brick Veneer/Steel Stud Wall Design and Construction Practices in Canada-Results of a 1986 Survey"), the authors' personal observations and overall review of this topic, the construction of these walls has preceded the development of adequate design, construction and inspection standards. This situation in turn has led to many sectors of the building industry being concerned over the long term safety, serviceability and durability of this form of construction.

It is the aim of this publication to discuss critical design, construction and inspection issues which may affect the long term performance of both the BV/SS and BV/CM wall systems.

The authors acknowledge with thanks permission granted by CMHC and W.G. Plewes to include portions of text and selected diagrams from the 1981 Advisory Document.

The work reported on in this publication was funded under a contract with the CMHC Project Implementation Division. The authors wish to thank the CMHC Project Manager, R. Duncan, as well as the Technical Advisors to this project for their assistance and interest in this study. The Technical Advisors were D.A.E. Fowler, Architect, Halifax; A.J. Garwood, Consulting Structural Engineer, Ottawa; T.W.J. Trestain, Consultant, Toronto; W.G. Plewes, Consultant, Winnipeg; P. Zirpke, Masonry Contractor, Vancouver.

## CHAPTER 3
## INTRODUCTION

### 3.1 Background and Scope

In Canada today two types of backup systems for brick veneer and cavity wall construction are commonly employed in highrise construction. The two backup systems are the concrete masonry or block wall and the steel stud forms of construction. While block backup has been widely used for decades, the newer steel stud backup system came on the Canadian market only in the 1970's. When CMHC in 1981 published the Advisory Document "Exterior Wall Construction in High-Rise Buildings", the steel stud backup system was still very much in its infancy in Canada and hence the publication emphasized issues related to the block backup system.

A recent CMHC sponsored survey of brick veneer/ steel stud building practices found that design and construction practices differ widely and that both the designer and contractor seek guidance about best design and construction practices. Clearly there is a need for an updated Advisory Document which addresses all important design and construction issues for both the brick veneer/ concrete masonry (BV/CM) and brick veneer/ steel stud (BV/SS) enclosure systems. This then is the overall goal of the present publication. Sub-goals include the following:

- discussion of the rain screen principle and its relation especially to the BV/SS system
- documentation of best current design, construction, and inspection practices
- presentation of common deficiencies and remedial measures
- presentation of suggested details
- listing of relevant code and standards requirements.

### 3.2 Environmental Conditions

For the long term satisfactory performance of cladding systems, differing environmental conditions must be taken into account in the choice of materials, design details, as well as construction and inspection requirements. Differing environmental conditions which come into play across Canada include moisture, wind, temperature (especially temperature fluctuations resulting in a large number of freeze-thaw cycles) and corrosive agents such as salt spray in a marine environment or certain industrial pollutants.

In the absence of more detailed guidelines to enable the designer to cope with different environmental conditions which can significantly influence the long term performance of enclosure systems, it is recommended to

- at least consider that differing conditions exist and that, for example, material requirements and design details may have to be significantly different for instance for a project in Halifax as compared to Calgary
- check local performance records
- assess the potential of increasing corrosive conditions in the future as, for instance, industrial development introduces more corrosive environments in a given region.

### 3.3 Advisory Document Overview

An overview of the range of topics and scope of the Advisory Document can be obtained from the Table of Contents presented in Figs. 3.1 to 3.3.

## PART 1 - INTRODUCTION TO STUDY AND TO VENEER WALL SYSTEMS



Fig. 3.1 Advisory Document Table of Contents

Fig. 3.2 Advisory Document Table of Contents

Fig. 3.3 Advisory Document Table of Contents

# CHAPTER 4
# CAVITY WALLS AND VENEER WALL SYSTEMS

## 4.1 General Description and Definitions

Masonry cavity walls and masonry veneer wall systems share the features of an exterior wythe of brick or other unit masonry material which is separated from an inner wythe or a backup wall by an unfilled vertical space or cavity. While it is often difficult to distinguish between these two types of walls either by appearance or behaviour, and also while in practice the terms are often used interchangeably, their differences should be understood. Specifically, a veneer is a non-loadbearing facing attached to a structural backing or backup but not relied upon to act with the structural backing in resisting lateral and vertical loads; for cavity wall construction on the other hand, the wythes are relied upon to act together in resisting lateral loads and additionally may act together in resisting vertical loads.

For purposes of this publication, veneer and cavity walls will be treated identically for all requirements other than structural and will hence be referred to simply as veneer walls.

The veneer typically is supported at each floor level by a steel shelf angle or reinforced concrete slab. In the case of a steel structural frame, steel shelf plates may occasionally be employed.

Summarizing, this Advisory Document will deal with design, construction and maintenance issues pertaining to brick veneer vertically supported on a

- steel shelf angle or
- reinforced concrete slab

and laterally supported by non-loadbearing

- concrete masonry backup or
- steel stud backup.

Both backup systems have their strong and weak points. Table 4.1 provides an overview of some key advantages and disadvantages for both systems.

## 4.2 Veneer Wall Requirements

In common with other forms of exterior wall construction, veneer walls must fulfill basic requirements pertaining to

- aesthetics
- durability
- air barrier
- vapour barrier
- fire resistance
- acoustic barrier
- thermal resistance
- structural strength and stiffness.

The feature which sets veneer wall construction apart from other forms of construction is the use of the rain screen principle to resist rain penetration and to vent moisture.

## 4.3 Functions of Veneer Wall Components

Components of the two veneer wall systems are shown in Figs. 4.1 and 4.2.

Table 4.1 Advantages and Disadvantages of CM and SS Backup Walls

CM Backup

| Advantage | Disadvantage |
|---|---|
| • satisfactory performance over many years | • large selfweight |
| • large stiffness which can readily resist full lateral load from wind and earthquake | • significant floor space requirement |
| • continuous wall system for support of insulation, air/vapour vapour barriers, and interior finishes | • installation of electrical services within CM wall somewhat complex |
| • not subject to corrosion or other deterioration | • if non-loadbearing, inherent stiffness requires side gaps to accommodate inter-storey drift of structure |

SS Backup

| Advantage | Disadvantage |
|---|---|
| • low selfweight | • longterm performance history not yet available |
| • construction relatively independent of weather conditions | • relatively flexible for support of stiff veneer leading to potential veneer cracking |
| • relatively small floor space requirement | • detailed design required for numerous components and connections |
| • electrical services easily accommodated within SS wall | • requirements for frequent inspection because construction details become quickly hidden |
| • insulation placement within SS wall | • metal components, particularly connections for ties, may be susceptible to corrosion |

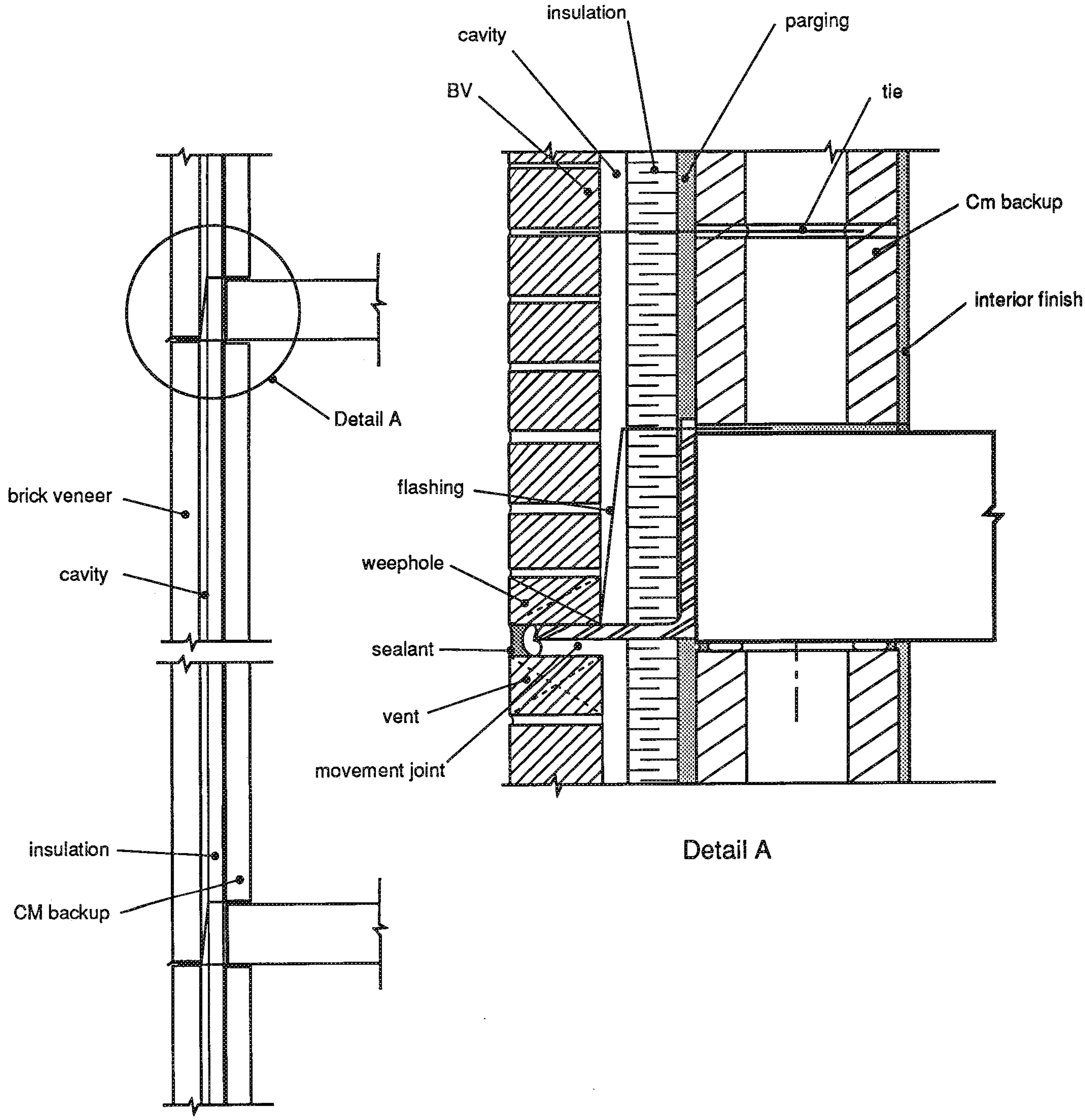



Fig. 4.1 BV/CM wall system components.

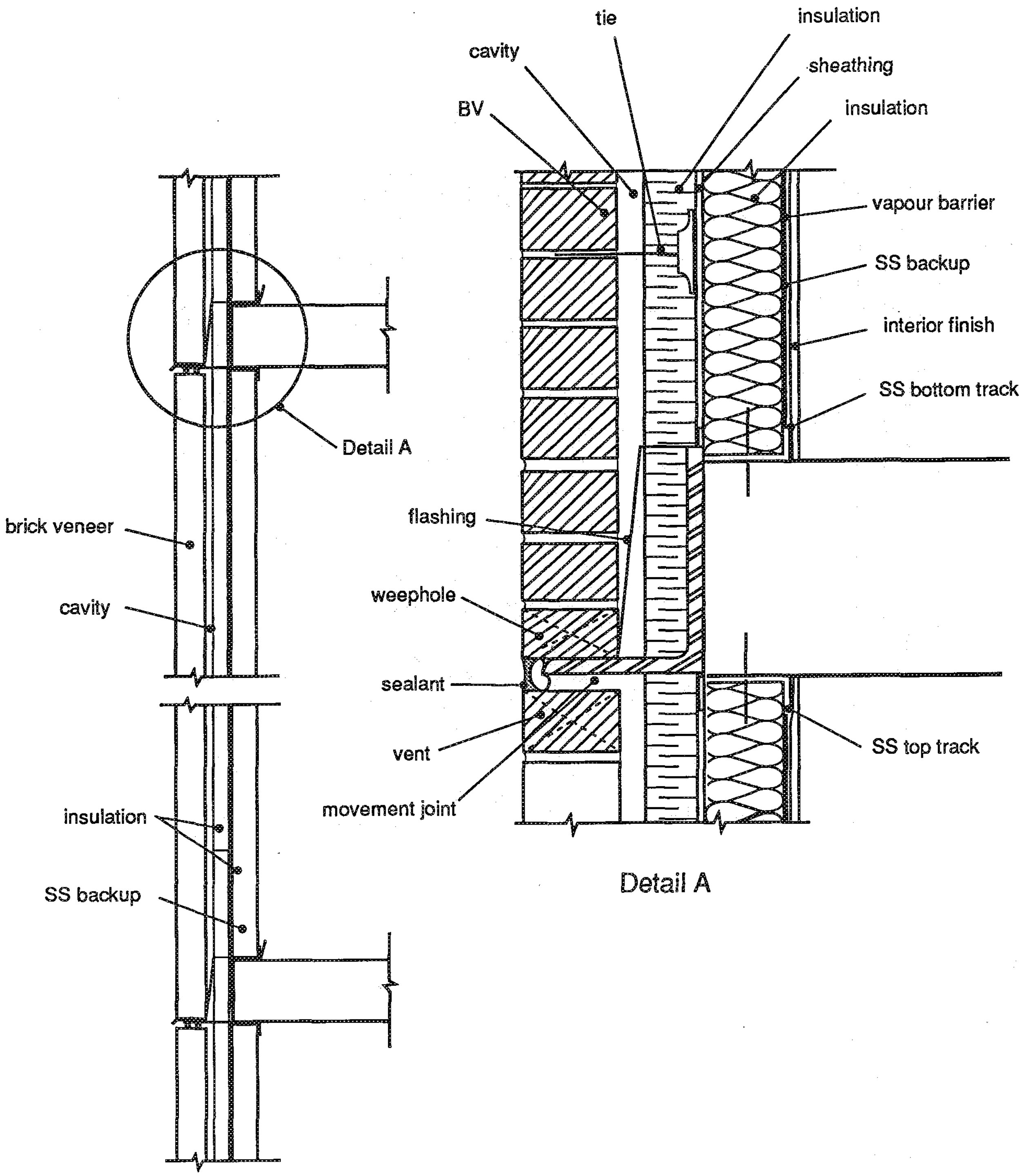



Fig. 4.2 BV/SS wall system components.

# CHAPTER 5
# GENERAL DISCUSSION OF STRUCTURAL REQUIREMENTS

## 5.1 General Design and Construction Requirements

The structural design of BV wall systems should satisfy these general requirements:

- adequate strength to resist design loads
- adequate stiffness to assure serviceability under normal conditions
- adequate durability including compatibility of materials
- adequate robustness not to be adversely affected by normal construction and use conditions
- adequate redundancy and toughness to withstand damage without completely collapsing
- adequate detailing of the components of the wall system and the interconnections between them to assure that the wall performs as intended
- integration with both the behaviour of the supporting structure and the building science requirements for the wall system.

Some of these considerations, as they generally apply to brick veneer wall systems, are discussed in the following sections.

## 5.2 Strength

As is illustrated in Figure 5.1, the backup wall system for brick veneer must be designed to carry its own weight in the vertical direction and to resist out-of-plane lateral load due to wind, earthquake, and general occupancy and use.

As will be discussed in more detail later, the structural requirements for the veneer itself are less well defined since, by definition, the backup wall is designed for all of the lateral load. However, serviceability and interaction with building science related performance can be used to help define strength criteria for the BV where cracking may be considered a type of failure. As is shown in Figure 5.2, prior to cracking of BV, the share of the lateral load resisted by the BV is much greater where the backup is relatively flexible such as is typically the case for SS backup walls.

Specific aspects of loading as applied to BV wall systems are discussed briefly.

<u>Wind Loading</u>

Wind pressure normal to the wall most frequently controls its design. As is set out in the National Building Code, the design pressure depends on geographic location, exposure, and internal pressure conditions. Regarding the latter, breakage of windows and failure of doors or vents during high winds will cause internal pressures in the building which should be added to the corresponding positive or negative external wind pressures.

For wind loads, the National Building Code of Canada provides a full treatment of pressure distributions over the faces of buildings and the effects of wind gust and exposure conditions on both internal and external wind pressures. However while internal pressures are always resisted directly by the backup wall because the air barrier is located at this position, the location of application of external pressure is not always clear.

Theoretically, the use of the veneer as an open rain screen implies that airflow through vents and weep-holes results in the cavity having the same air pressure as the outside surface of the veneer. If such is the case, as is shown in Fig. 5.3, the veneer is in equilibrium and the backup wall has the entire load applied to it. However, even in this case, the stiffness or resistance of the veneer to bending will result in ties redistributing part of the wind load back to the veneer as indicated in Fig. 5.4. As mentioned previously, the magnitude of this redistribution depends on the relative stiffness of the veneer and backup wall and on tie stiffness. If ties are very stiff, then the veneer will have nearly the same deflection profile as the backup.

In actual fact, full pressurization of the cavity may not be achieved because:

- air leakage through the backup wall limits pressurization of the cavity
- an insufficient volume of air can be moved through openings in the veneer to equalize short term pressure changes due to wind gusts
- as illustrated in Fig. 5.5, movement of air in the cavity to areas of unbalanced pressure (i.e. sidewalls compared to windward walls) may make it impossible to move a sufficient volume of air through vents and weepholes
- pressure differences over the face of the wall mean that cavity pressure will differ from external pressures over most of the wall area
- vents and weepholes are omitted, partially plugged or simply too small to transmit sufficient air for cavity pressure equalization.

In the extreme, nearly the full force of the external pressure may be transmitted by the veneer. While analyses indicate that this is the most critical loading case for maximum tie forces and bending moments in the veneer, variations in the relative stiffnesses of the walls and the tie stiffnesses and locations can result in significant changes in these internal forces.

Seismic Loading

For seismically active areas and particularly where relatively massive walls are used, out-of-plane forces proportional to the mass of the wall can control design. This loading can be especially critical for connections for exterior wall elements where the horizontal force factor, Sp, is 11 in the 1985 NBC. Depending on the value of the zonal velocity ratio, v, and the weight of the wall, Wp, the lateral force, Vp = v Sp Wp can be quite large. Fig. 5.6 contains a sketch and calculations illustrating this effect for a 3m height of brick veneer.

In addition to the strength approach to the design of the brick veneer wall systems, good engineering design practice should include some consideration of performance under damaged conditions and general resistance to complete collapse.

## 5.3 Serviceability

Serviceability requirements for structural design are very much related to the many building science aspects of design. However, clear definition of acceptable criteria either explicitly or through arbitrary limits are generally lacking.

## 5.4 Durability

A key feature of structural design is to assure that the "as constructed" components will retain characteristics reasonably close to their original properties. This means initial strengths and stiffnesses should not deteriorate over the life of the structure, otherwise design provisions must take into account anticipated changes. Example effects which should be considered include fatigue or repeated loading, general loosening of components, corrosion, freeze-thaw damage and wetting. In addition to assessing the potential influence of these types of changes on structural behaviour, their interactions with building science design requirements are also important.

For aspects such as corrosion, it should be recognized that climatic and geographic factors can lead to large differences in the potential for corrosion. Construction in industrial areas may lead to the rain being slightly acidic whereas in some geographic locations, such as along the east coast, rain may frequently be accompanied by high winds which lead to greater leakage. Consideration of such factors along with the intended or likely life of the structure may show that to allow gradual deterioration is not acceptable and in this example the use of non-corroding system components such as ties may be essential to assure the long term safety of the wall system.

## 5.5 Robustness and Redundancy

The design should incorporate a robustness which will ensure that normal construction practices and normal use will not damage any components or significantly reduce their ability to perform the intended functions. In addition, good design will incorporate a toughness and degree of redundancy which will enable the wall system to withstand damage without completely collapsing.

The creation of toughness or redundancy in a design involves ensuring that failure of a single component will not result in collapse. Some degree of ductility and/or alternate load paths can usually be incorporated at minimal cost and often at no cost at all. Examples of robustness and redundancy are shown in Figure 5.7.

## 5.6 Importance of Details

Details should be clearly thought out in terms of all the related functions. They should be clearly drawn and described in the specifications, and they should be sufficiently simple to achieve compliance during construction.

## 5.7 Shop Drawings, Samples and Calculations

SS backup construction requires production of shop drawings. Similarly but to a lesser extent, anchors, joints, and openings in CM backup must be completely detailed. These drawings should include all necessary details for assemblage and erection including installation of windows and other attachments.

## 5.8 Materials Handling and Storage

All masonry materials at the construction site, such as masonry units, sand, lime, and cement should be protected against wetting prior to use by storing on elevated platforms and covering with waterproof sheet material. In winter, sand should be stockpiled in such a manner that heating is possible if necessary. Steel reinforcement, ties and anchors should be kept free of oil, dirt, and heavy rust which may impair bond. Ties should be handled with care so that no mechanical damage occurs.

Steel studs should be stored indoors in their wrappings or containers until ready for use. Because of relatively thin sections, the studs can be easily bent, dented or twisted; care in handling is therefore necessary. If the panels are first assembled on the floor, it may be necessary to lift the assemblage at two or more points depending on the length of the panel.

## 5.9 Inspection

Experience has shown that it is normally best practice for the designer to be responsible for inspection of the brick veneer wall system. Responsibility for inspection should be clearly set out in the contract documents and, where other than the designer is assigned this responsibility, knowledgeable and proven experience with this type of construction is essential.

The specifications or other contract documents must clearly set out schedules for inspection and requirements for the contractor to provide notice of completion of specific parts of the construction before proceeding to close in or otherwise cover completed work. Too often either random or fixed times for inspection are ineffective means of quality assurance.

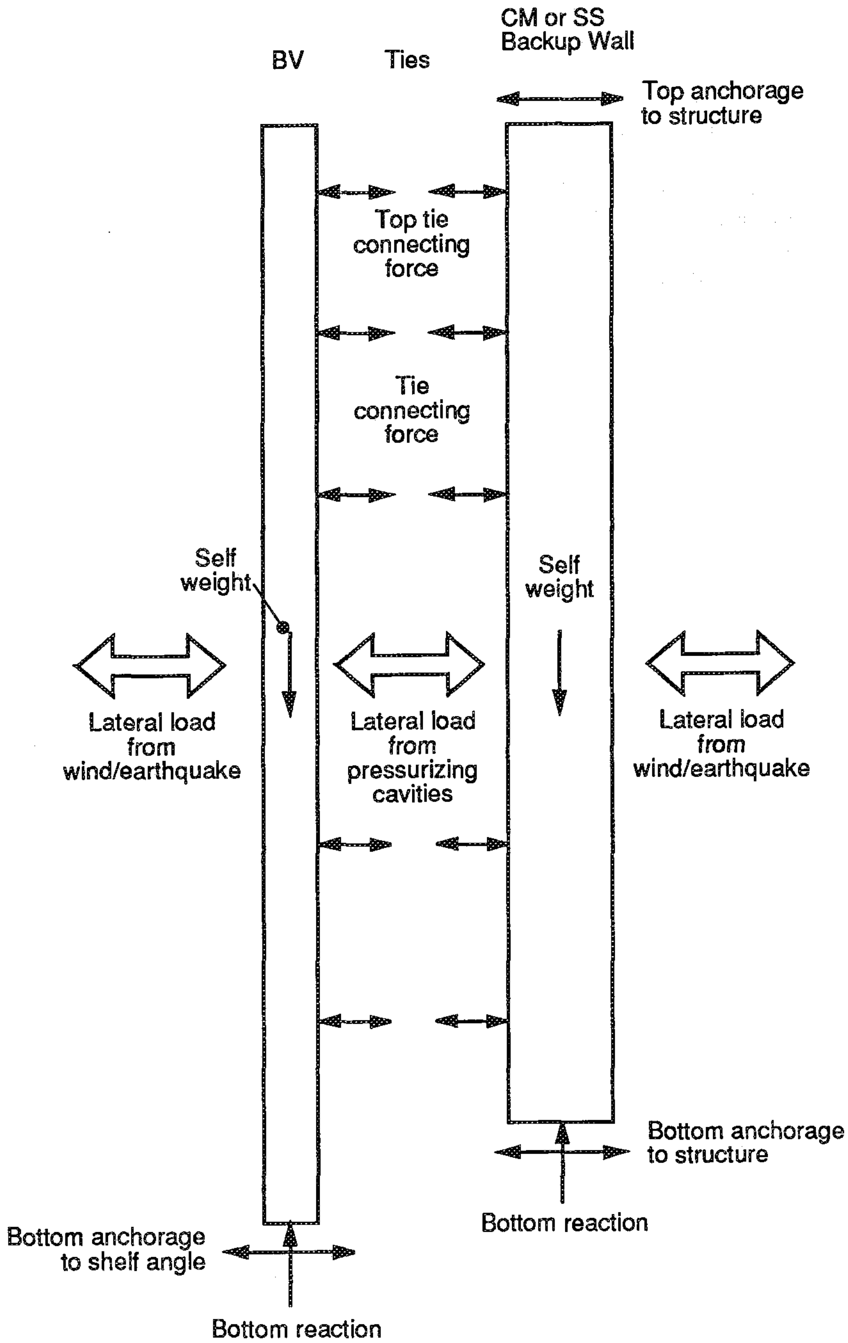



Fig. 5.1 Schematic drawing of forces on BV wall systems

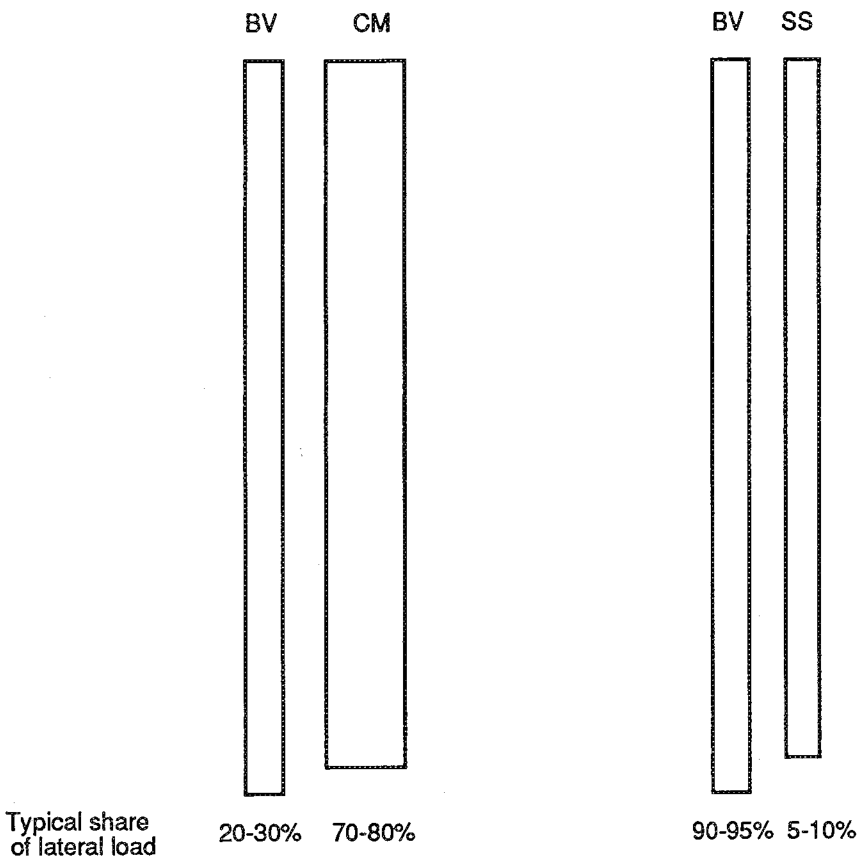



Fig. 5.2 Typical load sharing between BV and backup walls for uncracked BV.

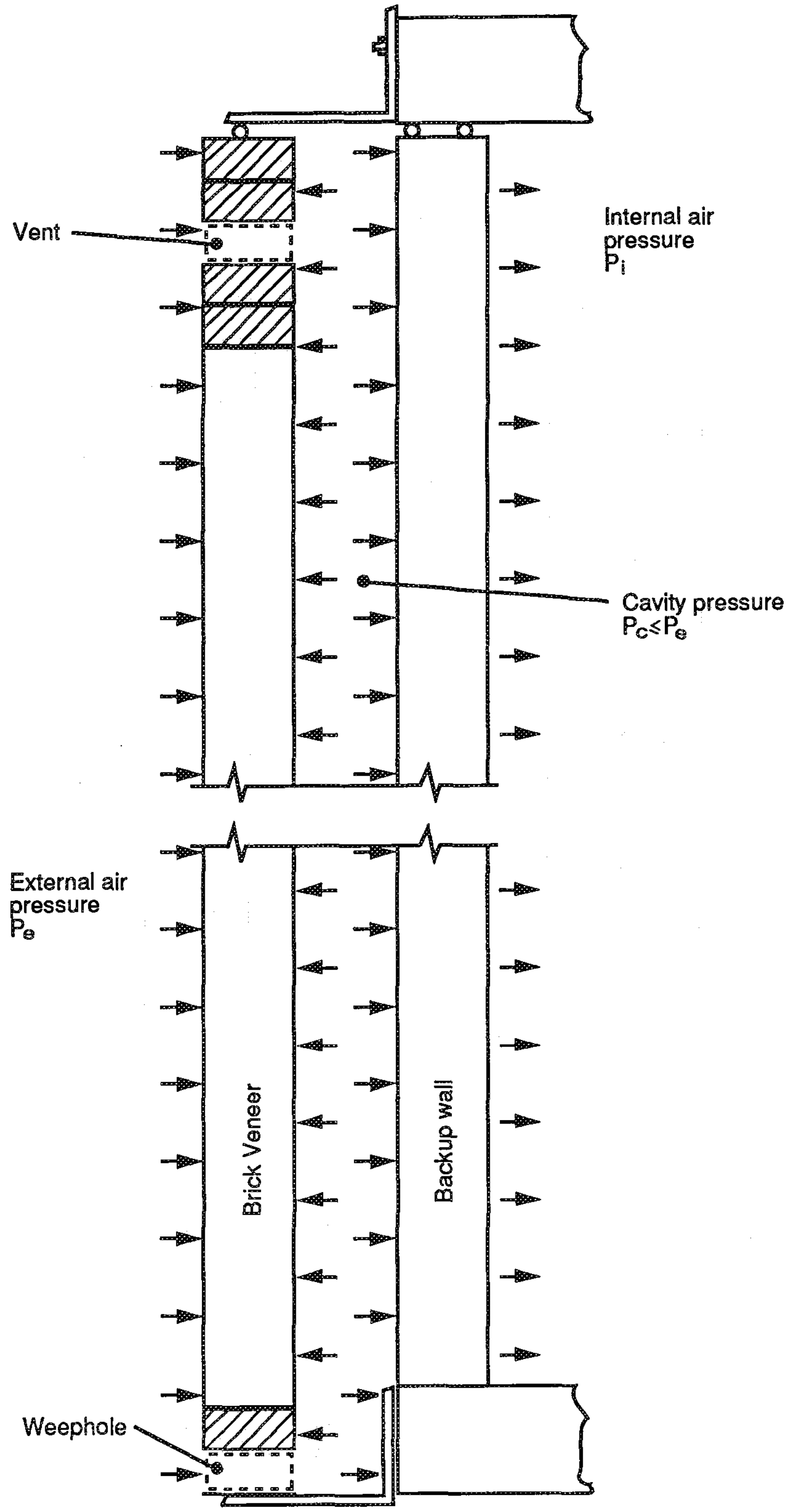



Fig. 5.3 Distribution of wind load to brick veneer and backup wall for pressurized cavity case.

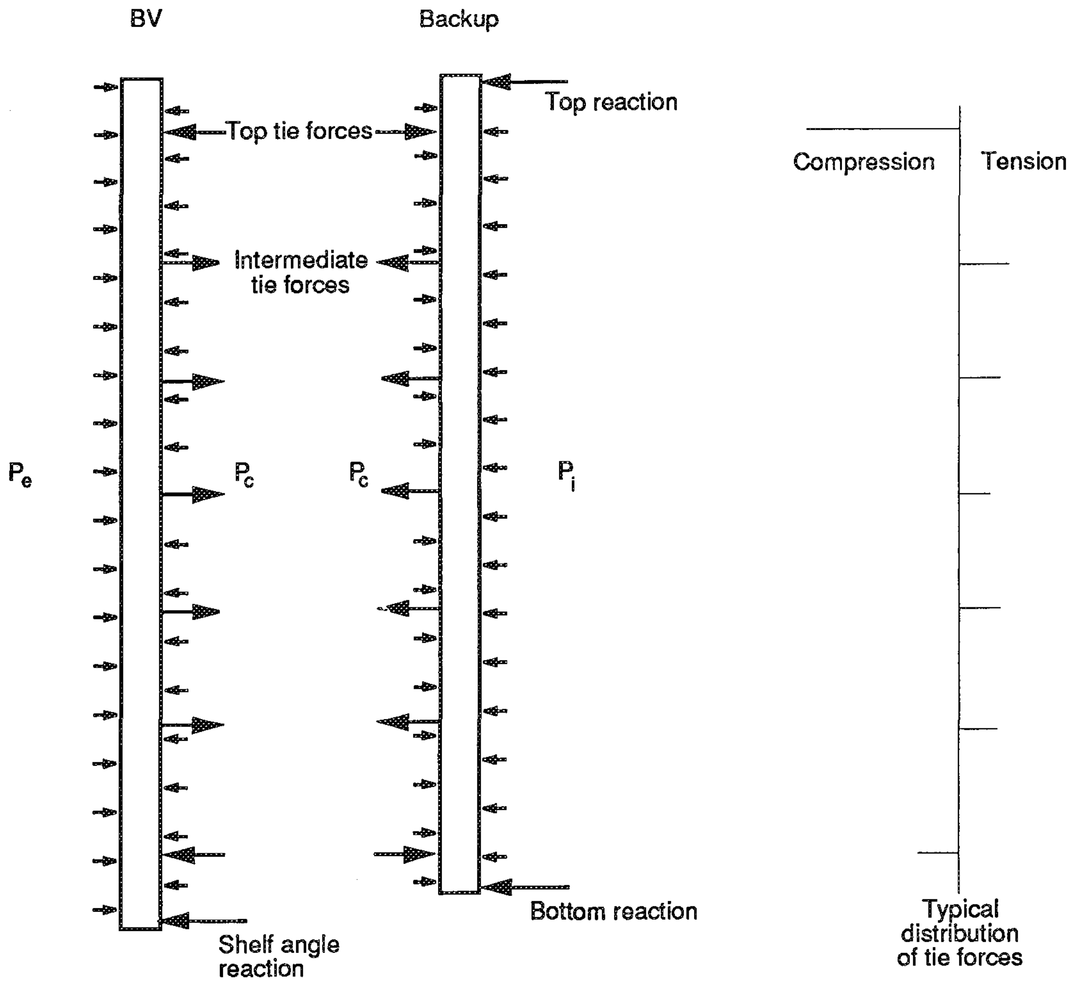



Fig. 5.4 Redistribution of load back to BV through tie forces.

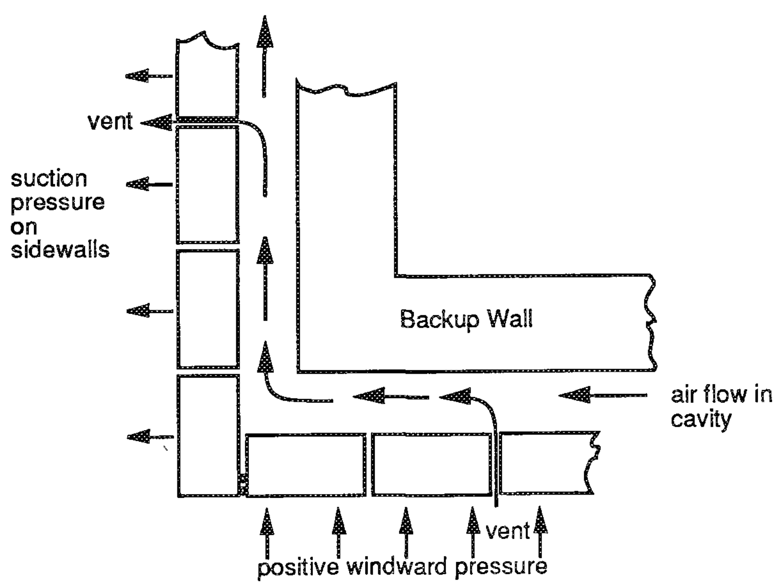



Fig. 5.5 Lack of pressure equalization in cavity due to air movement.

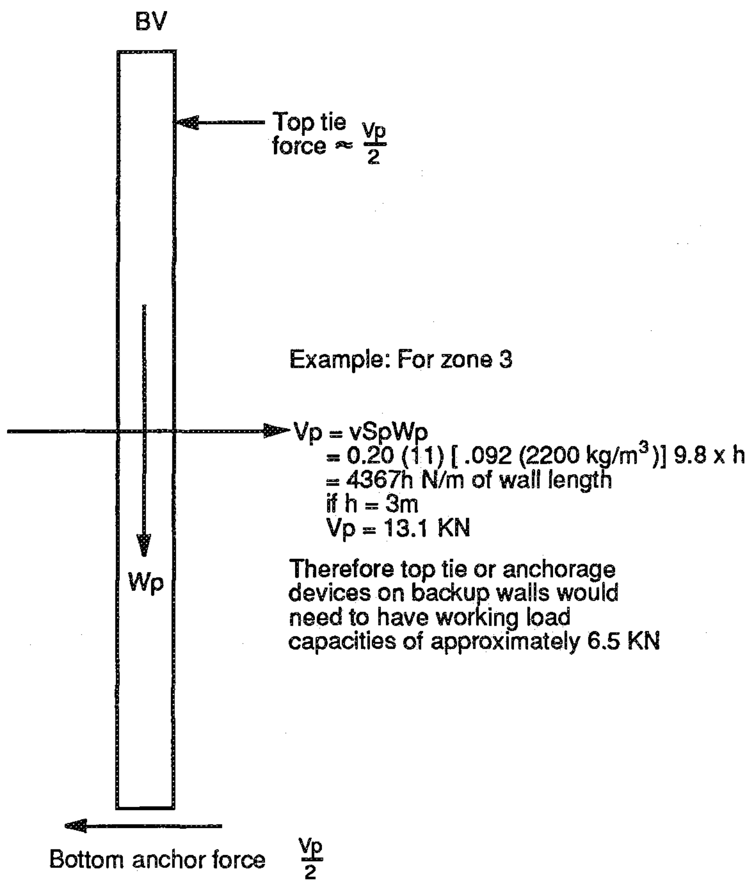
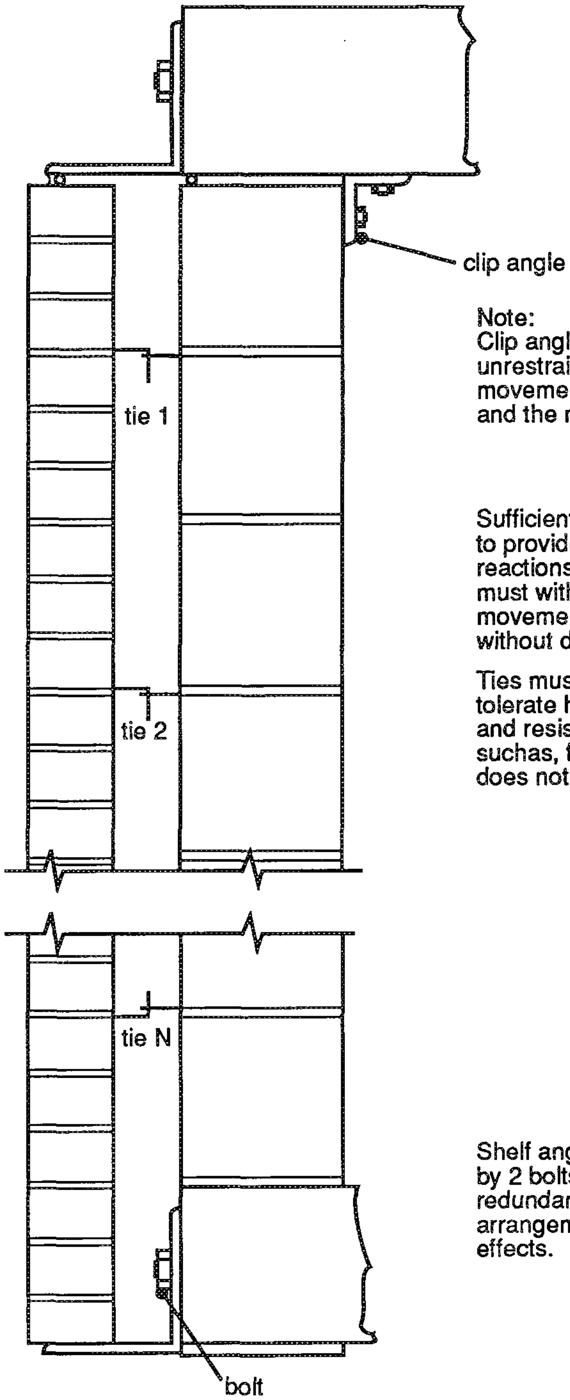



Fig. 5.6 Example calculation of connector forces for earthquake loading

While Tie 1 may be adequate to resist applied loads, inclusion of ties 2 to N provides redundancy which, aside from assuring more uniform distribution of load, reduces the importance and consequences of failure of any individual tie.

In a situation where damaged veneer may tend to fall away from the building, ties 1 and 2 provide a certain toughness by retaining load resisting capacity wheras tie N would tend to disengage and not add toughness to the construction, A tie which cannot disengage, even under extreme displacement would be preferable




Note:
Clip angle detail must allow unrestrained differential movement between the slab and the masonry wall.

Sufficient clip angles must be used to provide a redundant line of reactions. The methods of attachment must withstand repeated loading, movements, and environmental forces without deterioration.

Ties must be sufficiently robust to tolerate handling on the job site and resist damage during installation suchas, for instance, where coursing does not match.

Shelf angle segments anchored only by 2 bolts do not represent redundancy or toughness. A three bolt arrangement would provide both effects.

Fig. 5.7 Examples of robustness and redundancy

# CHAPTER 6
# STRUCTURAL COMPONENTS COMMON TO VENEER WALL SYSTEMS

## 6.1 Introduction

This section contains the requirements for the structural design of the common components of BV wall systems.

## 6.2 Backup Wall

The interior wall has been called the backup wall because it normally provides the structural backing for the brick veneer. As such it has been traditionally designed to resist all of the lateral load and to transfer the lateral load to the main structure of the building.

The backup wall can be designed independently from the brick veneer. For loading, a uniform distribution of the total load is a reasonable assumption. (Note: Where the brick veneer is more than 10% higher than the backup wall, the design load on the backup wall should account for the potentially greater tributary area.) The capacity requirements for this backup wall will be affected by whether it spans vertically, horizontally, or both, and by the extent of openings and methods of transferring forces from these areas to the supporting structure. For large windows or other openings, significant increases in the wall's load carrying capacity may be required on either side of the opening.

For CM walls which are built into moment resistant frames, the inter-storey drift can result in in-plane forces being exerted on the wall. Therefore, unless the backup is designed as infill to be part of the load resisting structure, appropriate gaps (in the order of span/500) are required along the sides. Connections to the columns must permit this movement. Where the main structure is a shear wall structure, the inter-storey drift is usually sufficiently small that SS backup walls can be built in tight to the side supports. The more rigid CM backup may still require some clearance.

Despite the fact that the brick veneer may not be relied upon to share in resisting lateral load, because it is connected to the backup wall means that it does share in resisting the load. Since the strength and stiffness of brick veneer cannot simply be "legislated out of existence", the veneer and backup wall will actually function as a cavity wall up to the point where the stiffness of the veneer changes due to cracking after which load sharing is less significant.

Since the veneer and backup wall may not be of equal height or length, nor have the same support conditions, nor be tied together by rigid ties, calculations which distribute the load to each wythe in proportion to their structural section stiffness (EI) will at best be very approximate.

In summary, the backup wall will normally be designed to carry the full lateral load and, in the case of a stiff backup, to be sufficiently stiff to prevent cracking of the veneer. Alternatively, where a relatively flexible backup wall is used, it should be designed to limit the overall deflection of the wall system to within acceptable values; at the same time, it is recommended that the brick veneer have sufficient strength to carry most of the load up to some level at which cracking is considered acceptable.

## 6.3 Brick Veneer

Brick veneer provides a pleasing aesthetic appearance, partially resists external loads and acts both as a direct and indirect moisture barrier as part of the rain screen principle. Brick veneer is expected to perform these functions better if it remains uncracked. Therefore the design criteria must set out the conditions under which cracking is acceptable.

For cases when the only feasible design is to allow cracking at some stage of loading below the full design load, it is still worthwhile to assess the level of load at which cracking is expected. In fact, specification of some minimum load to be carried prior to cracking could serve as a lower bound serviceability limit. Since in many cases walls are never subjected to the full design load and since secondary strengthening effects are often present, it is probable that many such walls will not crack in service. Although special provisions are necessary to accommodate the increased water penetration due to cracking, the occurrence of rain penetration related problems will be reduced due to the smaller proportion of walls which crack.

It should be noted that the support conditions have marked effects on the cracking capacity of brick veneer where support around all sides of a panel will significantly increase the cracking load. Since ties and anchors transfer load between the brick veneer and the backup wall and/or main structure, the stiffness and location of these also affects the cracking capacity. Conservatively, for the cracking load, the veneer can be designed independently using simple support conditions and applying the portion of the load calculated on the basis of relative EI values. That is

$$P_{bv} = P_{total} \frac{EI_{bv}}{EI_{bv} + EI_{backup}}$$

where

$P_{total}$ = total uniformly distributed lateral pressure

$P_{bv}$ = uniformly distributed lateral pressure applied to the brick veneer.

If the backup wall or main structure does not provide unyielding support, evaluation of the load carrying capacity of the veneer by itself is a conservative alternative to a rigorous analysis.

Computer programs such as a plane frame analysis can be used for structural analysis of BV wall systems. The results of these analyses provide reasonably accurate information which allows detailed examination of the effects of design decisions.

Where cracking of the brick veneer occurs, much of the load on the veneer is transferred to the backup. Especially for high walls, the potential for additional cracking must be considered where instability of the veneer could result from a crack spacing at less than twice the tie spacing. Figure 6.1 illustrates this feature. Since the location of cracks in the veneer also affects the tie forces, analysis of potential crack patterns may be required for unusual wall geometries.

## 6.4 Vertical Support for Brick Veneer

There are basically three ways in which veneer is supported at its base.

1. The most common method is for the brick veneer to be supported on a shelf angle as shown in Fig. 6.2.

2. The veneer is supported directly on the floor slabs of the building.

3. For loadbearing masonry backup, construction practice in some areas has been to support the veneer on the foundation wall and build the brick veneer continuously for the full height of the building.

## 6.5 Structural Requirements for Shelf Angles

As illustrated in Fig. 6.3, the applied moment is the weight of the veneer multiplied by the distance from the face of the support. The resisting moment is the tension force times the distance to the compression component of the couple. Therefore it is very important that the correct type of shims be used to provide horizontal adjustment.

A major decision in the design of the shelf angle is to what extent its position can and should be made adjustable. If the shelf angle is fixed in position at the time of concrete placement for the structural frame, its location both vertically and horizontally is tied to the construction tolerances of the frame. Such a method of shelf angle attachment presents some advantages and disadvantages:

Advantages:

- Attachment to the frame, usually by means of a cast-in strap anchor, is secure.
- The construction is simple and straightforward and hence can proceed with a minimum of potential slip-ups and a minimum of site supervision.

Disadvantages:

- Because of construction tolerances, variation in the vertical position of the shelf angle can result in either too large or too small a gap for the movement joint beneath the angle.
- Variations in the horizontal position of the shelf angle can only be accommodated by varying the bearing area of the brick on the shelf angle.

Alternatively, an adjustable shelf angle attachment is possible by means of cast-in anchors or drilled-in anchors. This attachment method has the following advantages and disadvantages:

Advantages:

- Vertical adjustment is possible by means of slots in the shelf angles.
- Horizontal adjustment is achieved by means of shims.
- Good aesthetics are provided due to even joint widths.

Disadvantages:

- Cast-in anchor bolt locations may not match shelf angle slots.
- Drilling-in of new anchors or enlarging slots in the shelf angle are common means of correcting mismatched slots.
- Greater construction care and inspection effort are required to assure proper alignment of the shelf angle, proper use of shim plates, and proper torquing of the friction connection created by tightening the nuts on the anchor bolts.

## 6.6 Ties

The adoption of CAN3 A370 Connectors for Masonry by the National Building Code in 1985 has had the significant beneficial effect of focussing attention on the need to evaluate both the structural and long term performance of ties. This standard provides guidance for methods of evaluating ties and the type of safety factor which should be associated with various potential modes of failure. However, the actual strength and stiffness requirements are not directly specified, and tables for horizontal and vertical spacings infer a uniform tie force or tributary area concept for ties which is not correct. Both full scale tests and detailed structural analysis have shown that the top tie fastening veneer to the backup wall is by far the most highly loaded and for flexible backup walls this load may approach half of the load on the wall. Therefore, ties must either be designed for these higher forces or closer horizontal spacings near the top of the brick veneer.

From a structural stiffness point of view, some of the ties available are inadequate. Of concern, other than the flexibility is the free play which some forms of adjustment (i.e. oversized holes) incorporate. Also, as shown in Figure 6.4, ties should be capable of carrying load even in cases of extreme deformation.

The indention or drip which had been incorporated into many types of ties to prevent water from flowing across the cavity has been eliminated in order to provide stiffer and stronger ties.

Although 22 ga. corrugated ties are allowed in the NBC for cavity width up to 25 mm, variations in bends in the ties can have very large effects on tensile stiffness and compression buckling strength. Therefore use of corrugated strip ties is not recommended for highrise construction.

Where screws or dissimilar materials have been used, corrosion has been a major concern for the long term properties of the ties. Therefore ties which were once only mill galvanized now are hot dipped with some designers requiring stainless steel or other non-corroding materials especially under aggressive environmental conditions.

Ties connecting the brick veneer to the backup should be chosen to satisfy the conditions discussed in the following section.

Adequate Strength and Stiffness: Ties should be capable of transferring both tension and compression forces. Although in the past the tie forces have been considered reasonably evenly distributed, this has been shown to be untrue. Flat, corrugated ties should not be used. Where two piece adjustable ties are used, the wire part of the tie should be embedded 50 mm into the veneer. Also ties should not have mechanical play in excess of 0.8 mm and should not deform over 1.2 mm at 0.5 KN load in either tension or compression. Test data including strength and deformation information for the full range of adjustability should be required before any tie is specified for use.

Adequate Corrosion Protection: Since deterioration of ties may lead to an unsafe veneer, the corrosion potential should be carefully evaluated for the building environment.

Suitability for Methods of Construction: Sequence of construction and requirements for precise layout are major factors in the choice of tie system.

Suitability to Design of the Wall Components Including Non-Structural Components: Ties must fit in with the overall construction scheme recognizing the stage of construction and functions of the components connected to or otherwise interacting with the ties.

Sufficient Robustness to Withstand Handling and Use Under Job Site Conditions: Ties which may be easily damaged (bent, welds broken, corrosion protection removed) during handling are not suitable for job site conditions.

## 6.7 Anchors

In the context of this Advisory Document, anchors are devices which are used to attach the backup wall and/or brick veneer to the supporting structure (for instance using dovetail anchors to concrete columns). Therefore, in the case of the backup walls, they must be capable of transferring the horizontal shear forces. In addition, at least at the top of the wall, the types of anchoring systems must allow unrestrained vertical movement at the movement joints.

CSA Standard CAN3 A370 "Connectors for Masonry" covers design and construction for a variety of anchors including so-called standard anchors.

## 6.8 Openings

The backup wall must be designed to carry the load transferred to it by the framing around the opening and that framing must be securely anchored into the backup wall. The degree to which framing is attached to the veneer depends on the strength and type of frame. However, in most instances the joint between the frame and the veneer should be a movement joint and the interaction between veneer and backup should be provided only by the extra ties around openings.

## 6.9 Key Construction Requirements

From the preceding discussion and additional considerations, the following key requirements for construction stand out:

- Prior to constructing the BV and the backup walls, the alignment, plumbness, and spacing of floors, columns and other components must be within acceptable limits.
- For shelf angles supporting veneer:
  - The vertical alignment of floors or edge beams should be sufficiently close that the overhang of the veneer with respect to the toe of the shelf angle is within accepted tolerances.

- For bolted connections, field adjustments by shimming or torch cutting must be approved by the Engineer.
- Where large variations in vertical alignment exist, drilling in of new anchors or replacement of shelf angles may be necessary to provide a smooth and even surface over the height and width of the building while retaining the specified cavity width.
- When field adjustments are made, touch-up painting or other approved procedures must be used to re-establish the corrosion protection.
- In the case of cast-in anchors, lack of adjustability makes it especially important that existing construction be measured up and problems resolved prior to installing the wall system.
- The provision of the specified clear movement gap at the top of the veneer is essential. Shelf angles must be firmly anchored to the structure to maintain the gap and mortar must be kept out of this space.

• For slab supported veneer, the tolerance related to slab edge versus veneer overhang is very tight. Not only must the veneer have adequate bearing but the top of the veneer must fall sufficiently under the slab to be protected from direct exposure to rain water running down the face of the building. Some of the construction requirements are illustrated in Figure 6.5. Although a popular detail in some parts of Canada, leakage problems, have reduced its use in other areas.

• Installation of ties must assure
  - Proper embedment of ties and joint reinforcement in mortar.
  - Damaged ties must not be used and ties installed must not be bent or otherwise damaged during construction. Figure 6.6 contains sketches of some concerns regarding tie placement.
  - The spacing for ties must be adhered to and particular attention must be paid to providing extra ties around openings and to assuring that the topmost row of ties is installed. Figure 6.7 illustrates some specific points requiring attention.

• Lintels over windows, doors, or other openings must be entirely contained within the veneer and not attached to the backup wall or structure. If vertical movement joints coincide with one end of the lintel, provision to allow movement must be incorporated as shown on the design drawings.

• The backup wall must be anchored to the structure at its base and sides and/or top but also must permit horizontal and vertical movement joints to function as intended.

• Brick must satisfy Canadian standards and should be chosen from those with proven service records. Use of glazed brick is not recommended.

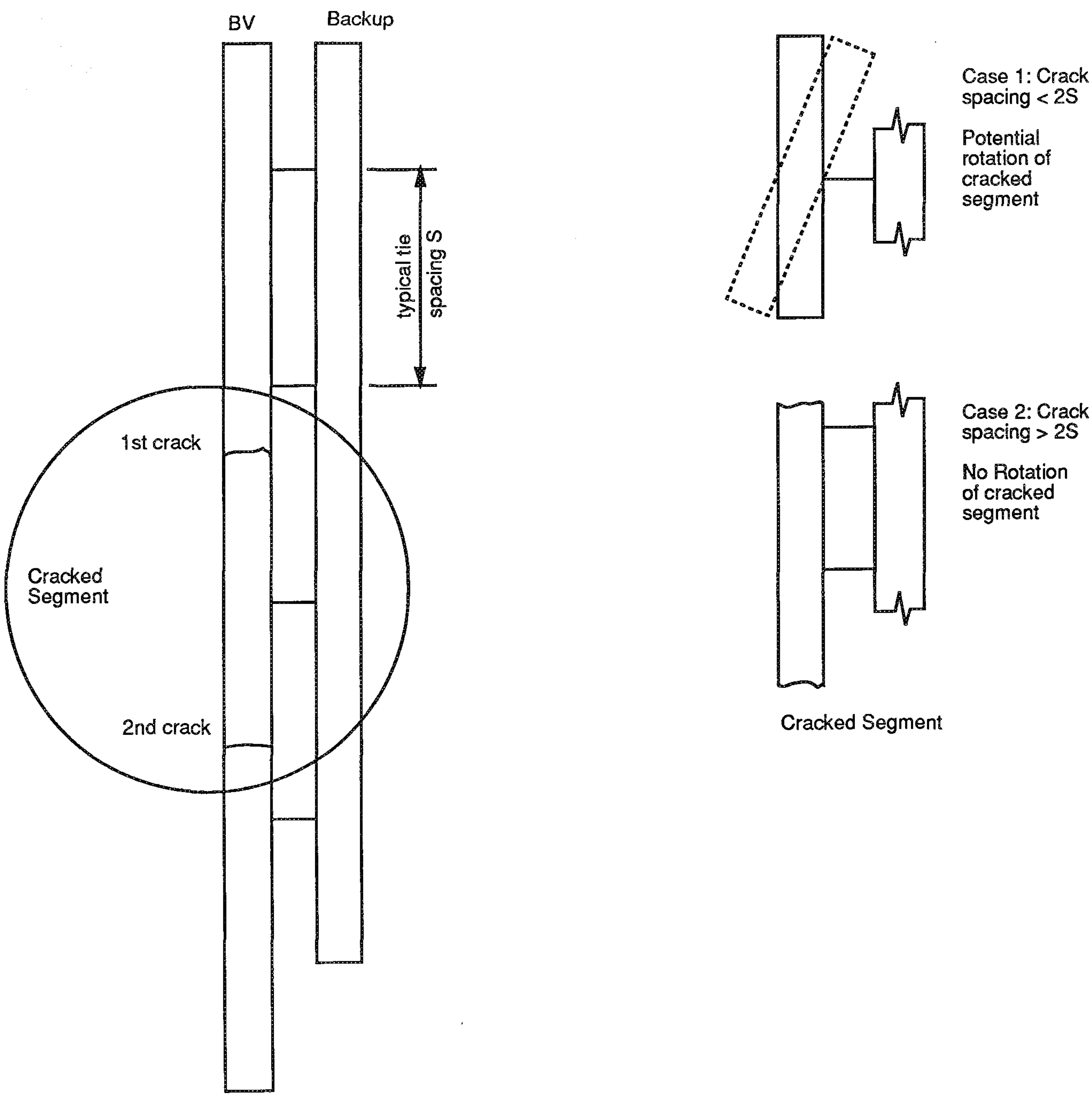



Fig. 6.1 Illustration of potential instabilty of cracked veneer for large tie spacing

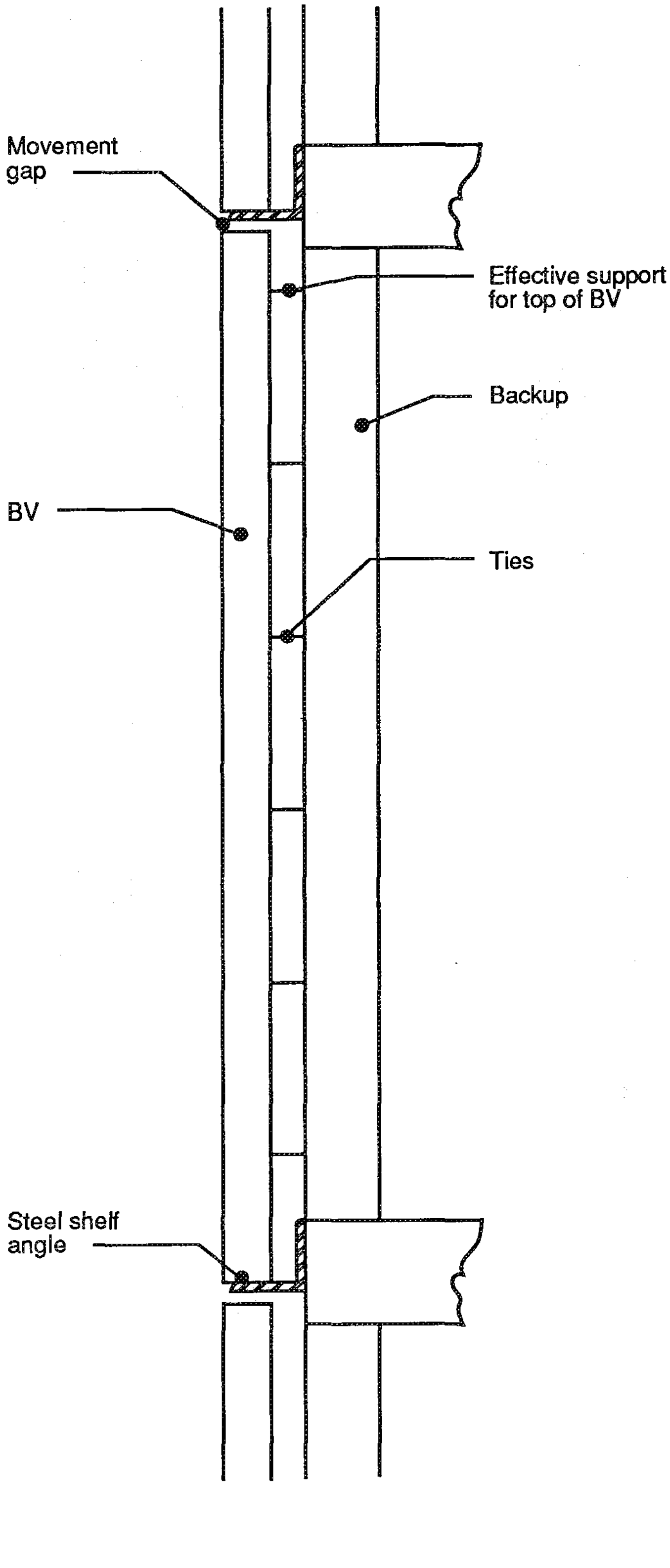



Fig. 6.2 Shelf angle support system for BV

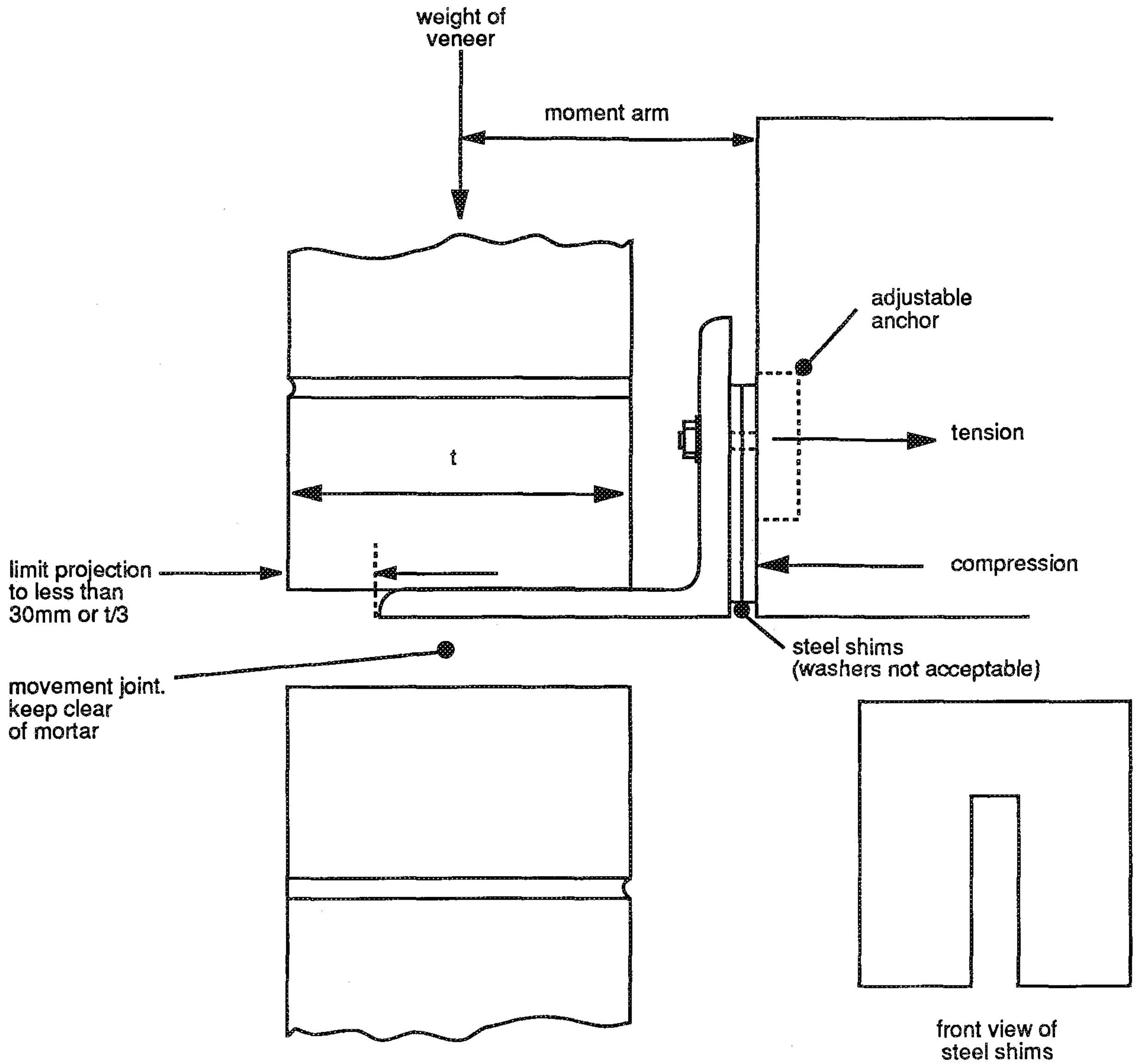



Fig. 6.3 Design of shelf angles.

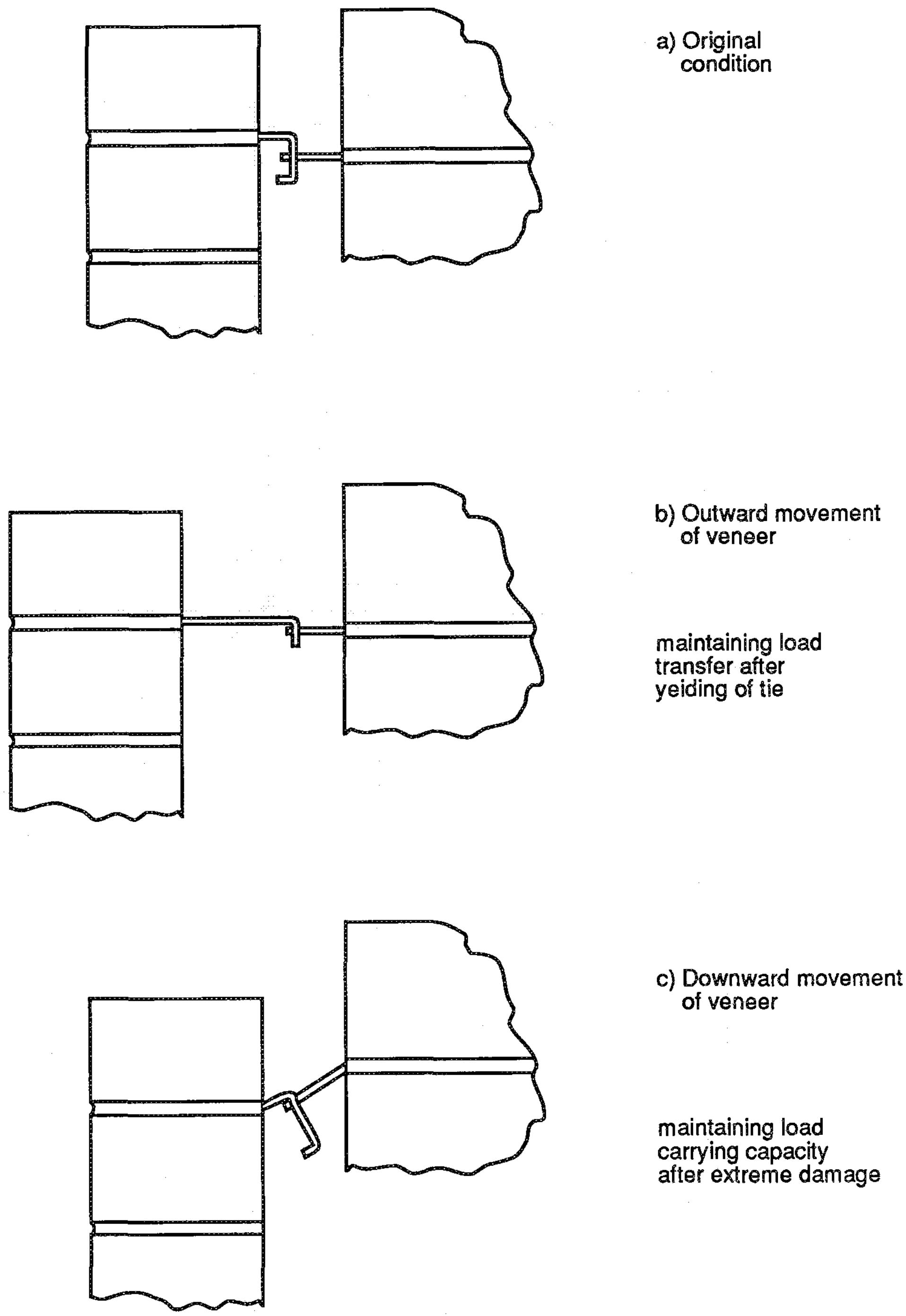



Fig. 6.4 Illustration of requirement for ties to maintain load carrying capacity after initial failure.

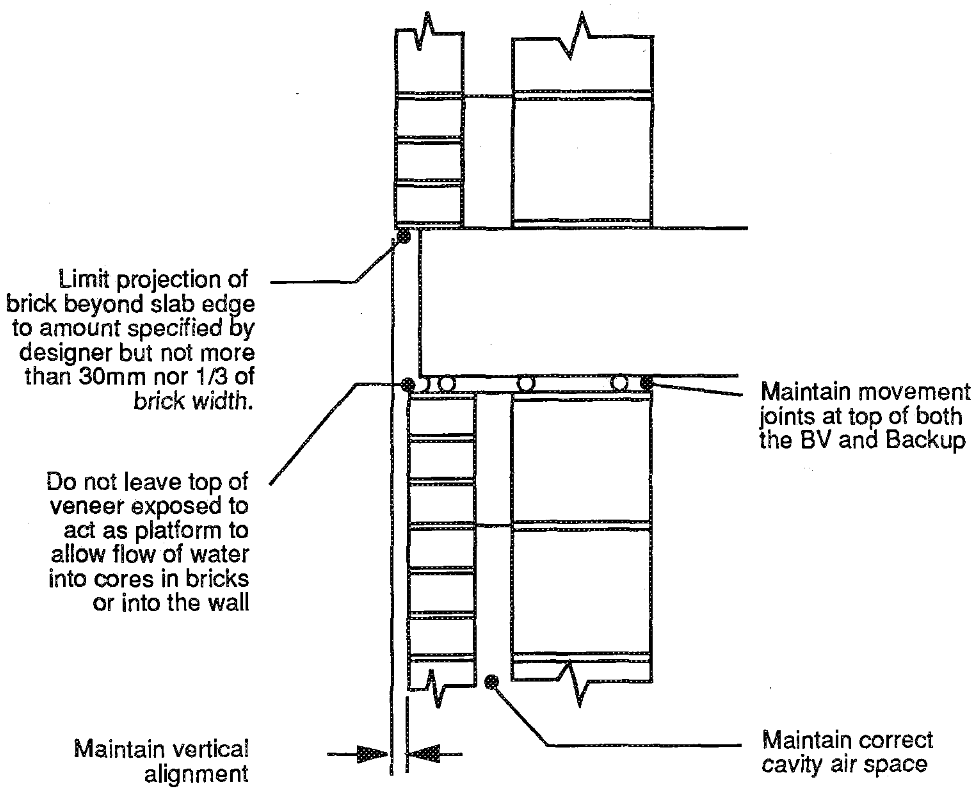
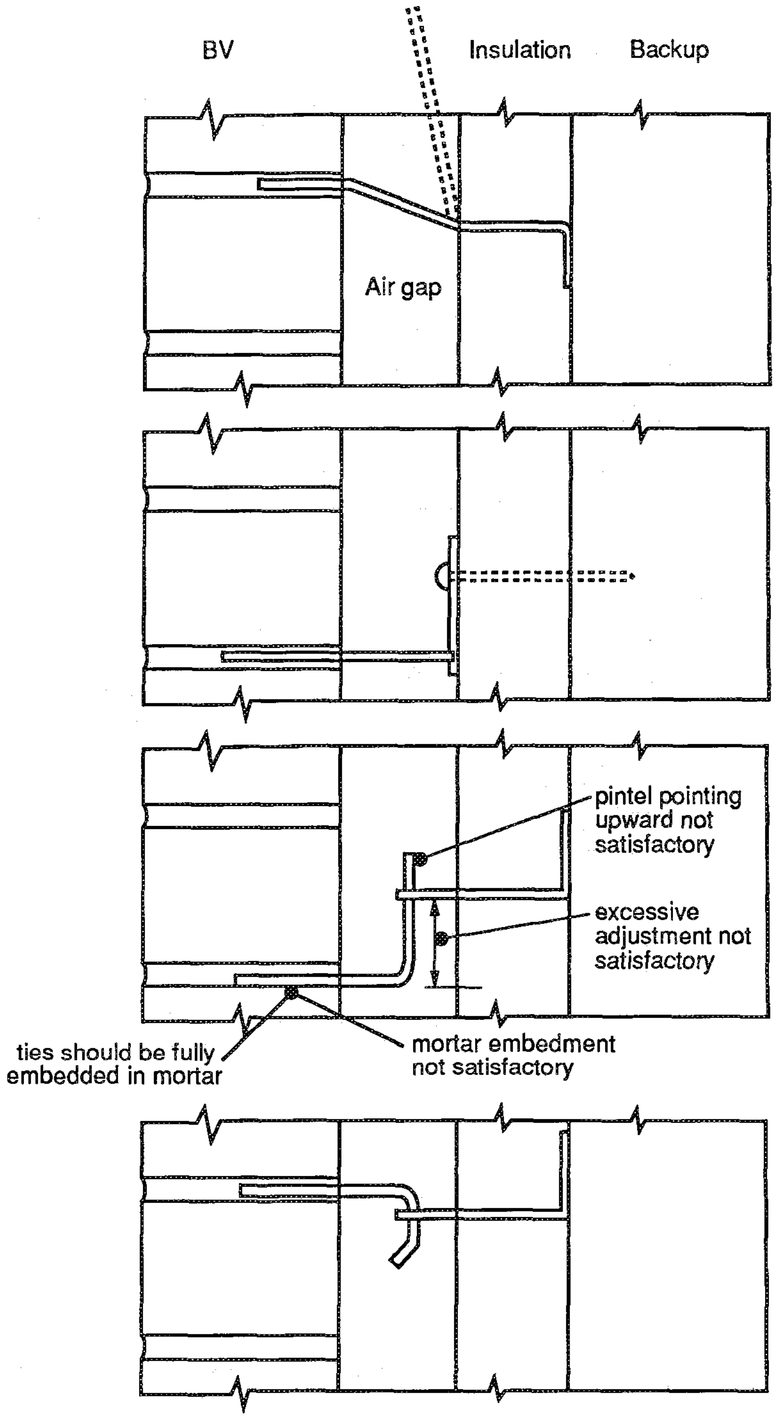



Fig. 6.5 Illustration of tolerance related issues for construction of BV on floor slabs




Ties should not be bent out of place during construction. Damage to the tie or misalignment when bent down may substantially reduce capacity

For use of ties intended to be mounted on insulation or other wall coverings, data must be available to demonstrate satisfactory performance.

Adjustable ties should not be able to disengage under extreme load/ displacement. In this case the pintel should point down. The adjustment should be minimized.

Satisfactory construction detail showing proper tie embedment, small adjustment, and pintel pointing down.

Fig. 6.6 Construction concerns regarding tie placement.

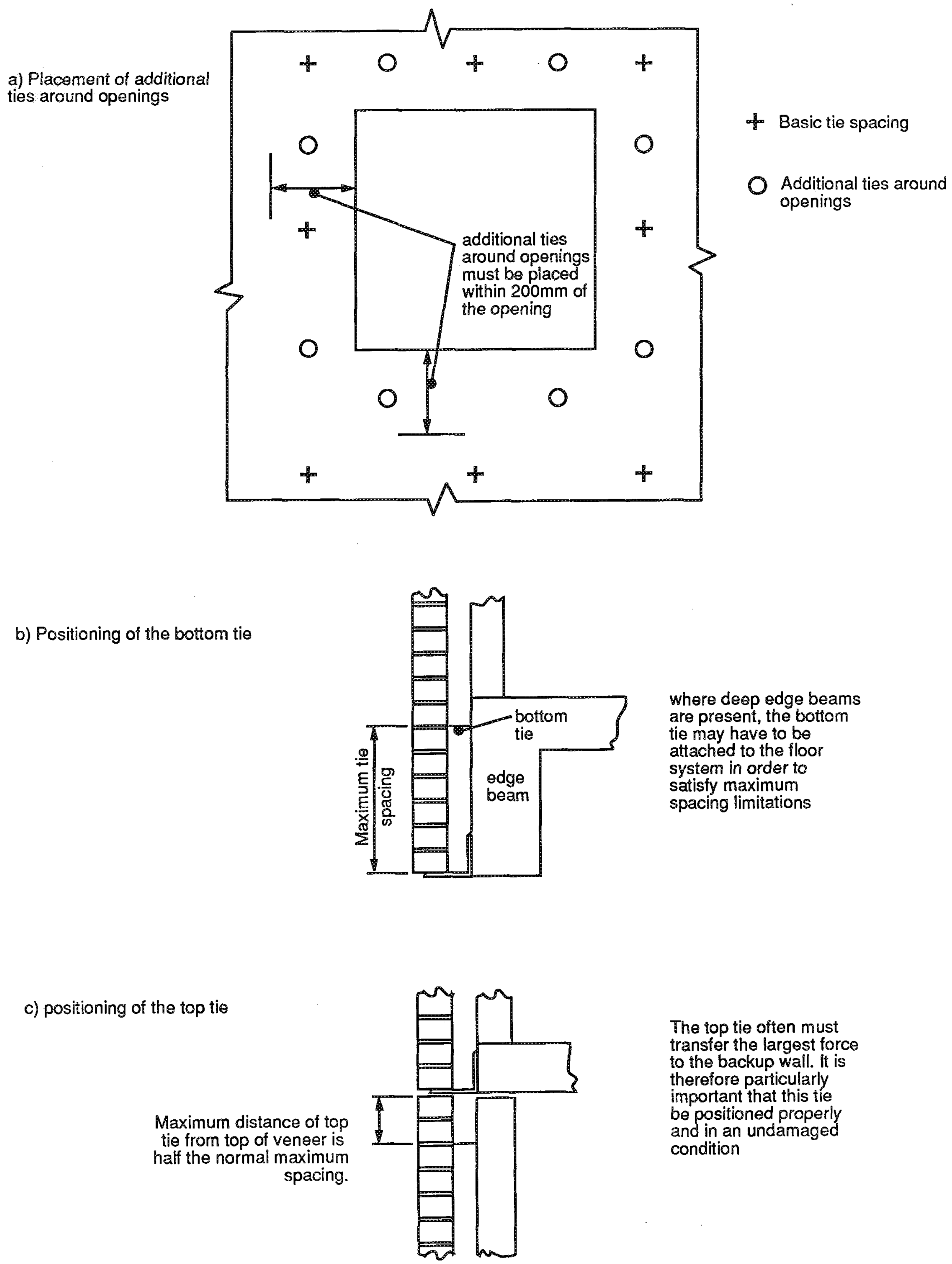



Fig. 6.7 Special considerations for tie placement.

# CHAPTER 7
# BV/CM SYSTEM: STRUCTURAL REQUIREMENTS

## 7.1 Introduction

The provisions for the design of brick veneer with concrete block backup walls are contained in CSA Standard CAN3-S304 for loading conditions discussed in Chapter 5. While the application of these provisions requires some interpretation, only a few key comments will be included here.

## 7.2 CM Backup

The CM backup wall must be designed to resist lateral loads, but often this is done using the Empirical Rules for Plain Masonry of Section 6 of CAN3 S304 where height to thickness limits of 20 are specified with minimum thicknesses of 140 mm and 190 mm depending on whether the block is a solid or hollow unit. Users of this design method must ensure that the limitations for its use are satisfied. Otherwise plain CM backup may be designed by engineering analysis to withstand the wind pressure or seismic load.

In some cases, it is desirable or, as is the case for Seismic Zones 4 or greater, it may be necessary to reinforce the block backup. In Seismic Zones 4 or greater the minimum amount of steel of 0.0010 Ag must be placed in 2 directions at a spacing not exceeding 1.2 m and with a distribution which is a minimum of 1/3 of the steel in any direction. In Seismic Zones 3 or less the minimum of 0.0005 Ag may all be placed in one direction provided that it is continuous between lateral supports, is located to adequately reinforce the wall against lateral loads, and the spacing does not exceed 400 mm where joint reinforcing is used. Typically, additional reinforcement equivalent to at least a 15 M bar must be introduced around all openings.

## 7.3 Brick Veneer

For typical 90 mm brick veneer, the maximum lateral load shared by the veneer prior to cracking has been found to be between 10 to 30% for 20 cm and 15 cm blocks depending upon the relative moduli of elasticities. Therefore, prior to the block wall cracking due to flexural loads the tensile bond stresses in the brick veneer will be quite low. However for reinforced blockwork where cracking must occur to develop the necessary tension in the reinforcement, the backup wall will be much more flexible and there is greater likelihood of flexural cracking of the brick veneer. A conservative estimate of the percentage of load shared by the veneer can be calculated using the moment of inertia of the transformed area of the cracked section for the block and the moment of inertia for the uncracked brick veneer. The ratio of the EI value of the BV to the sum of the EI of the BV plus CM will give the proportion of load carried by the veneer.

## 7.4 Key Construction Requirements

The following key requirements for construction of the system stand out:

- Concrete block should be kept dry to minimize shrinkage of the concrete masonry.
- Specified mortar proportions must be followed to achieve proper masonry bond.
- Sufficiently fluid grout of about 250 mm slump is to be used and vibrated/puddled to assure complete filling of voids to be grouted.
- Specified placement of all vertical and horizontal reinforcement is essential to achieve design strength and in the case of cover, durability. This means proper splicing of reinforcing bars, lapping of joint reinforcement (typically 150 mm) and staggering of laps (typically 800 mm) are all essential.
- Proper tie placement is vital to the long term performance of the wall system. Of utmost importance are the anchorage of ties and avoidance of tie damage such as kinks which drastically reduce capacity.
- Proper anchor placement similarly is essential to the safe support of the backup to resist wind and earthquake loads.
- Movement joints must be kept clear of all materials which can inhibit free movement. The consequences of improperly constructed movement joints can be very severe.
- Unless specifically shown on drawings, joint reinforcement should not be continued across movement joints.
- Good masonry workmanship, including full filling and tooling of mortar joints, is required since the BV/CM wall system is exposed to demanding environmental conditions of loadings and moisture for many decades.
- No materials may be substituted without approval of the designer. In particular, ties must have the specified corrosion protection which must not be damaged during construction.

# CHAPTER 8
# BV/SS SYSTEM: STRUCTURAL REQUIREMENTS

## 8.1 Description of the Steel Stud Wall System

The typical components of a SS backup wall system are shown in Fig 8.1 where the steel stud is the basic member which must have top and bottom tracks for support and/or to frame the wall. In most designs some form of bracing or internal bridging is required to develop the flexural strength of the studs (ie. prevent flexural torsional buckling). The normal method of connecting these components is to use self-drilling short metal screws. However, welding has been used for some buildings and in particular where the SS wall system is prefabricated.

The structural design of the SS backup wall must include the requirements for attachment to the structure including allowance for movement and differential movement of both the structure and the backup wall. The effects of and interactions with sheathing materials also must be accounted for as similarly must the tie loads and influence of the brick veneer on the distribution and magnitudes of tie loads.

## 8.2 Design of the Steel Stud

The structural design of steel studs for wind or other lateral forces is normally governed by:

- maximum (midspan) bending moment
- specified out-of-plane deflection limit
- end shear.

In this regard most manufacturers' information is based on the assumption that full lateral support exists on both flanges of the studs (ie. no flexural-torsional buckling can occur). Also web crippling due to concentrated loads or reactions is not checked. Therefore as will be discussed later, both of these factors must be included as part of the design process.

Although tests have shown that drywall or other sheathing materials screwed to studs can result in composite action, repeated cycles of loading rapidly diminish the benefit of this composite action through enlargement of the screw holes or other loosening. <u>Therefore, it is recommended that screw-attached sheathing such as drywall not be relied upon either for flexural capacity or for increased section stiffness to reduce calculated deflections.</u>

The following specifier note:

> "Some sheathing materials such as gypsum drywall may lose their structural integrity when subject to a moist environment or when subject to a sufficient number of load cycles. Such materials may not be suitable to act as structural bracing. If the sheathing is utilized as bracing, it is standard practice in the industry to provide sufficient steel bridging to align members during erection and to provide the necessary structural integrity during construction as well as in the completed structure"

is from the "Lightweight Steel Framing Manual" CSSBI 50M-1987, August, 1987 and is also adopted in this Advisory Document as sound design practice. <u>Therefore typically metal bridging is required to assure development of the flexural resistance of the studs.</u>

Structural analyses and tests have shown that brick veneer will significantly reduce the bending in the steel studs prior to cracking of the veneer. However, unless the brick veneer is specifically designed and inspected to act as a structural member, the design of the SS backup wall must assume that the BV cracks and that the full wind or seismic loading is resisted by the steel studs. In addition, the interaction of the BV with the SS backup wall can result in very non uniform distributions of tie loads. As is shown in Figure 8.2 for a representative BV/SS wall prior to cracking, the top tie transfers a high fraction of the lateral load regardless of whether the load is on the BV or SS part of the wall. After cracking of the veneer, the tie(s) nearest the crack then transfers the majority of the load. The existence of these high concentrated tie loads not only affects the design of ties but also must be accounted for in the design of the studs.

The following comments relate to the key design criteria identified for SS members.

<u>Maximum Bending Moment</u>

Unless manufacturers provide certification that tests and analysis confirm that the full yield moment capacity of a section can be developed, designers must check this using the provisions of CAN3-S136-M84 Cold Formed Steel Structural Members. Where perforated webs exist, the influence of the openings on section properties and member behaviour must be accounted for. As mentioned above, the SS wall must be designed to resist the full lateral load. In this regard, the influence of eccentric tie loads and the concentrated loads from ties must be accounted for both in terms of member behaviour (torsion) and local behaviour (web crippling) in the vicinity of the concentrated loads.

In the case where the design of the SS wall is controlled by deflection rather than strength, application of the full requirements for discrete bracing in accordance with Clause 8.3.2 of CAN3-S136-M84 may not be necessary.

For studs spanning in the order of 2.6 m, tests have shown that a mid height brace and top and bottom restraint of both flanges as shown in Figure 8.3 may be sufficient. However, information pertaining to the particular tie used (eccentricity and degree of concentration of load) and section of the stud must be used in making this design decision.

Concentrated tie loads near cut-outs in the webs of studs must not be allowed. While proper detailing may be sufficient to avoid this, it is recommended that cut-outs in webs be limited to locations where they are required for internal bracing.

Splicing of studs to extend the length should be avoided. If required, the splice must be properly designed and detailed providing the specified strength and stiffness.

Deflection Limit

Deflection limits for the specified loads are normally based on serviceability criteria rather than strength. Aside from human sensitivity to deflection, the main criterion relates to size of cracking in the brick veneer. While a deflection of span/360 is currently the maximum acceptable value, other limits may be appropriate.

Where designers choose to adopt the criterion that allowable flexural tensile stresses in the veneer not be excluded, deflection limits between span/2000 to span/3000 may be required.

The deflection limits are normally based on allowing the steel studs to resist the total lateral load. Note that horizontal displacement of the SS wall due to local translation in the top and or bottom tracks adds to the overall deflection.

End Shear

To avoid premature failure of the stud at either the top or bottom, web perforations should be kept at least two times the depth of the studs away from the reaction points as shown in Figure 8.3. For flange connections, the steel stud should be attached to the track at both flanges. As will be discussed in Section 8.3, other methods of support can be incorporated into the design.

It should be noted that at openings such as windows, the studs on either side of the opening will normally be required to carry equal portions of the lateral load transferred from this portion of the wall, including stud wall areas above and below such openings. While use of double studs is generally good design practice, large openings (ie. more than three times the regular stud spacing) may require more than 1 additional stud on each side of the opening. These studs should be tied together by straps or other means to facilitate load sharing and where used, horizontal framing members above and/or below the opening should be capable of transferring the load to these studs. Figure 8.4 provides information on typical load distribution from window areas in walls.

Concluding Remarks

Use of 20 gauge steel studs has been common practice. However there is evidence that the apparent economics of using this lighter section in fact may not be substantial in terms of the overall cost of the backup wall. Experience with handling and shipping of 20 gauge studs revealed that many suffered some form of permanent damage prior to installation in the building. Such members must not be included in the construction. Alternately 18 gauge steel studs were much more resistant to handling conditions and very few exhibited any form of distress.

BIA Tech Note 28B recommends that 18 gauge be the minimum thickness for steel studs used as backup to BV walls. Following discussions with representatives of the steel stud manufacturing industry, this recommendation is adopted in this Advisory Document. Principal reasons for this decision are:

- tests have shown that current bridging and bracing practices are much more effective for 18 gauge studs
- effective screwed and welded connections are more easily achieved with the thicker gauge material
- screws connecting ties or other components to studs have increased contact area and are less susceptible to overturning or loosening
- the thicker material is less affected by corrosion.
- studs are less susceptible to damage prior to installation.

Industry information indicated that a price increase of approximately 15% of the total cost of the SS backup wall would cover the cost of changing from 20 gauge to 18 gauge steel. When allowance is made for discarding damaged 20 gauge studs, it was decided that the greatly improved strength and stiffness characteristics, durability properties, and buildability aspects associated with use of 18 gauge studs were very cost-effective.

## 8.3 Basis for Design of the Upper and Lower Tracks

Standard practice is to use upper and lower tracks to act as the reaction points for the steel studs. These tracks may also serve to:

- provide bracing against torsional buckling
- provide a continuous edge for caulking or otherwise sealing the wall against air flows
- provide a platform for support of drywall or other wall sheathing
- provide a movement joint (normally at the top track).

Common practice has been to use a thicker gauge of steel for the tracks than for the studs (ie 16 gauge tracks combined with 18 ga. studs).

In addition to assuring adequate strength, the stiffness of the design detail needs to be checked where lateral displacement might affect the performance of the veneer and where internal finishes might be damaged by local deformations. In lieu of a comprehensive analysis, it is recommended that under full service load, the local horizontal displacement at the track connection not exceed 1.2 mm. [A comprehensive analysis is recommended when cracking of the veneer or size of crack is particularly critical.]

Design of the Lower Track Connection

Stud to track connections are typically screwed so that the flanges are mechanically connected. This detail provides torsional restraint but does not transfer shear from web to web. Therefore the stud reaction force is transferred in bearing of the stud flange against the upstanding leg of the track. The track must be checked for local bending in the upstanding leg and as mentioned for stud design, web crippling at the end of the stud should be checked. Cut-outs near the end of the studs must not be permitted.

Because the veneer is typically supported by a shelf angle at its base, prior to cracking of the veneer the horizontal reaction at the bottom track may be relatively small. However as shown in Fig. 8.2, following cracking of the veneer or if ties are very flexible, the bottom track may be required to resist approximately half of the lateral load on the wall.

While the stud should be firmly pushed into the track to minimize the gap between the end of the stud and the web of the track, allowance for up to 6 mm gaps should be included in the design.

Welding of the stud to the track provides a superior connection for both strength and stiffness. Alternate details including web connections have also been used effectively.

Design of the Upper Track Connection

To ensure non-loadbearing behaviour, it is necessary that combinations of deflections and shortening of the loadbearing structure do not result in transfer of loads into the steel stud system. The size of movement joint required depends typically on the shortening of the structure per storey and on the long term deflection of the beam or floor system. While each building needs to be assessed individually, for normal storey heights and spans encountered in reinforced concrete construction, a 12 to 15 mm movement joint has been standard practice. For relatively flexible floor systems where long term deflections may be in the order of span/360, it is obvious that larger movements must be accommodated. In addition, a factor frequently not considered is the thermal expansion of the steel stud system due to fire. Taking this factor as well as construction tolerances into account, the size of a movement joint could exceed 25 mm. Obviously the provision of any movement joint has implications on the backup wall's stiffness and the requirement of a relatively large joint such as 25 mm would place added emphasis on this aspect of design.

From the previous discussion, it is clear that screwing in of studs to the track at locations of the movement joint is unacceptable. A number of movement joint details have been developed in practice to support the steel stud while permitting movement. Some are illustrated in Fig. 8.5 and will be commented on briefly below:

(a) FREE FLOATING STUD: As shown in Fig. 8.5 (a), this detail, one of the simplest in use, relies on the track only to provide out-of-plane support. Therefore bridging or internal bracing is required near the tops of the studs to provide torsional stiffening and in-plane restraint.

(b) NESTED TRACKS: This detail permits the studs to be screwed to an inner track for support. Then, as shown in Fig. 8.5 (b), by nesting this track inside another track having shorter legs, vertical movement is permitted. Because the studs have a more effective bearing area along the outside track, this detail generally is thought to provide greater stiffness than (a). Also, to a limited extent, the stud to inner track connection can be used to accommodate construction tolerances thereby reducing the size of movement joint required.

(c) WEB CONNECTION: A web connection of the type shown in Figure 8.5 (c) is sufficiently flexible in the vertical direction to allow free vertical movement while providing both in-plane and out-of-plane horizontal restraint. The advantage of improved out-of-plane stiffening can be coupled with a reduced requirement for leg height and gauge of track. Figure 8.5 (d) shows a similar detail but with sliding vertical movement accommodated by slots in the web connector.

(d) BOXED TRACK WITH FREE FLOATING STUD: This detail provides both in-plane and out-of-plane horizontal restraint for the end of the stud which is "free floating". Thus as is shown in Fig. 8.5 (e), it is able to accommodate the vertical movement. The boxes created by the nested, face to face tracks provide a relatively stiff support. The boxes should be filled with insulation to improve thermal resistance but, as is the case with the nested tracks, significant thermal bridging can occur with use of this detail.

Construction

As is illustrated in Figure 8.6 the most common error in construction of movement joints is that movement is inadvertently prevented by some feature of the construction, often resulting from incomplete design detailing. While screwing the stud to the track is the most obvious violation of free movement, attachment of drywall to the track and to the stud is effectively the same thing. Similarly running the drywall the full height of the wall will also offer resistance to movement and mean that something has to give for movement to occur. In many designs the above features have not been sufficiently thought through.

Where a gap at the top of the wall covering is not aesthetically acceptable, provision of the movement joint at the bottom of the wall may be a practical solution where the joint and the movement can be concealed behind baseboards or other trim. While this entails "hanging" the wall from the upper track, this has some structural advantages of reducing the displacements of the top tracks significantly by providing a stiffer connection. The more flexible connection associated with the movement joint will result in less displacement at the bottom of the wall due to generally lower reaction forces.

Where tracks are discontinuous over the length of a panel, two alternatives are available for providing continuous support:

1. Mechanical anchorage of the abutting ends of the tracks to the floor must be provided near both ends of the tracks.
2. Where the joint in the track is approximately mid way between studs, a 300 mm piece of stud may be used as a splice. Double screws on each leg of the track should be used to assure adequate continuity.

Tracks may be conveniently used as part of the structural framing above and below openings. Aside from assuring adequate flexural capacity, provision of a sufficiently strong and rigid connection to the adjacent studs is necessary. While various types of clip angles or other devices may be effective, an efficient and effective connection can be achieved by cutting the end of the track and bending the web as shown in Figure 8.7. This type of connection will resist torsion on the stud rather than cause it. Maintaining a proper movement joint does require that construction tolerances be accounted for. While small variations in storey height can be accommodated in the size of the movement joint, larger deviations must be corrected by using different lengths of studs.

Concluding Remarks

The provision of an adequate movement joint is an essential component of steel stud system design since otherwise the system may become loadbearing at some time in the service life of the structure. The type of track connection must be chosen with care and with full consideration of potential interaction with other construction features. Construction must ensure that movement as intended in the design is in fact provided.

## 8.4 Bridging and Bracing

It is recommended that wall sheathing not be relied upon to contribute to the structural performance of the stud wall system.

The most common form of bridging is internal through-the-stud bridging where a channel type of member fits snugly in prepunched regularly spaced holes. The bridging member is placed vertically, then twisted sideways to fit snugly against the sides of the hole. This snug fitting may be one reason that positive connection of the bridging to the steel has often been omitted on jobs. However, when such a workmanship deficiency occurs the bridging provides very little benefit. The normal method of positive connection of the bridging to the stud is to use a clip angle as shown in Fig. 8.8 and connect to the stud and bridging either by welding or by using a minimum of two widely spaced screw connections on each piece as shown. The bridging should be continuous and where joined should be jointed using welding or a minimum of 2 screws on each side of a splice as shown in Fig 8.9.

For residential storey heights of 2.8 m or less, it has become common practice to use only one line of bridging at about mid-height of the wall where the studs are restrained against twisting at both the top and bottom. However tests clearly show that internal bridging at this spacing does not provide sufficient torsional bracing to develop the full flexural capacity of the studs.

Other than experimentally verified performance, there is little guidance available regarding the design of bridging or bracing for torsional restraint. However, for channel sections it is known that these torsional restraints should be connected so as to effectively restrain both flanges of the section. Therefore the method of connecting the internal bridging member to the stud can have a significant influence on its effectiveness. Use of thicker gauge clip angles and more efficient screw placement and/or welding have been shown to result in significant improvement.

An alternative to internal bridging is to use surface bracing where the channel legs of the braces have been cut out at the locations of the studs and are screwed to the studs as shown in Fig 8.10. Test have shown that this system works well but has the disadvantage of not easily accommodating uneven spacing of studs, although new cut-outs can be made and existing cut-outs strengthened if there are not too many of these odd measurements. Surface bracing channels are required on both sides of the stud. As was the case for internal bridging, full flexural capacity is not achieved with a single line of bridging for normal storey height stud lengths. Similar comments apply.

Particularly for the inside sheathing which should be less susceptible to wetting, it may be possible to provide attachments which will assure that this layer will remain effective as lateral support for the studs. Development of mechanical anchors which do not depend on bearing between screws and gypsum board may satisfy this requirement. If so, the requirements for interior side steel bracing may be relaxed as research information becomes available.

### 8.5 Anchorage to the Frame

Except where a web connected support member is used to transfer load from the steel stud to the supporting floor system or spandrels, the connection is by attachment of the track to the supporting floor systems or spandrels. The best connections for the track are drilled insert expansion anchor or cast-in-place anchors as shown in Fig 8.11. For these, the hole in the track should be equal to the diameter of the anchor and a washer should be used to help transfer force as the bolt is tightened. Various forms of concrete nails and shot-in connectors are available and have been used on many jobs. However caution is advised in their use. Firstly, these anchors must resist a peeling action as well as direct shear and should be evaluated for these forces including repeated loading. Also, the long term ability of these to hold when quite small lengths of embedment occur should be evaluated independent of the capacity requirements. Spacing of the anchors affects the deflection of the track and therefore should not exceed 800 mm unless design calculations indicate otherwise.

Adjustments in the location of the tracks can be used to correct minor variations in vertical alignment of floors. However as is indicated in Figure 8.11, the track should not be positioned beyond the edge of the floor unless tests and/or analyses show that lack of full contact over the web of the track will not adversely affect the performance of the BV/SS wall system. Similarly minimum edge distances for the mechanical anchors must be maintained.

### 8.6 Brick Veneer

For large spans for steel studs, calculations indicate that cracking of brick veneer may occur at relatively low horizontal forces. This means that substantially increased rain leakage is highly probable and in turn the potential for damage due to moisture is greater.

### 8.7 Ties

Many ties have been developed to attach veneer to the SS backup wall. In selecting a tie system the capacity and deformation behaviour of a tie itself, as well as the full effects of the range of adjustability including positioning over the stud should be taken into account.

At McMaster University, an experimental study of a wide range of tie types and loading conditions revealed an equally wide range of performance characteristics. For the full range of adjustability, some existing ties do not have adequate strength or stiffness to satisfy the highly non-uniform tie load requirements associated with SS backup walls. Even though CAN3-A370-M84 does not specifically cover adjustable ties, the load factors for various loading conditions are applicable. Therefore, it is recommended that reliable information on the strength and stiffness of a tie system (including attachment to the stud) for the full range of adjustability should be required in order to make a proper design based selection. Limits on mechanical play and stiffness similar to those suggested in BIA Tech Note 28B are recommended.

Particularly for Canadian climatic conditions, the implications of using a screw in tension to transfer the tie force to the steel studs must be considered. Critics of this system have coined the term "hanging by a thread" with some justification where:

- inadequate corrosion protection may lead to loss of pullout strength
- overturning, drifting, or other incorrect installation of the screw may seriously lower the pullout strength
- installation through drywall or insulation may allow loosening of the screw under repeated load applications
- dissimilarity in materials and the rough hole created by self-drilling screws may result in crevice corrosion around the hole in the stud
- lack of ability to inspect installation may result in screws not even engaging the studs.

The potential for corrosion will depend largely on the likelihood of repeated wetting of the screw with rain penetration or condensation of exfiltrating air being the likely sources. External insulation and/or other sheathing over the screws will reduce the potential for wetting. Alternately, two-screw systems, bolt and nut systems, or those which transfer tension by other mechanisms may be more suitable types of ties.

In many cases the most conveniently located (and sometimes the only feasible location) for ties is on the sheathing material fastened to the stud. While some design guides recommend moisture protection of gypsum board for instance as a means of protecting this bearing area from deterioration, tests have shown that even without any deterioration of the material, the capacity and stiffness are dramatically reduced due to indentation of the drywall material. Also the effect of screws loosening due to cycled load should be considered.

Extra ties in the vicinity of openings are required because of the added load which must be transferred between the BV and the SS wall in this region. CAN3-A371-M84 provides guidance on this subject.

## 8.8 Key Construction Requirements

From the preceding discussion of BV/SS structural requirements and other considerations, the following key requirements for construction of the system stand out:

- Bent, twisted and otherwise damaged steel studs must not be used.
- Tracks should be properly anchored. Any extension of the track outside the floor edge must be approved by the designer.
- Movement joints at a track must provide the specified gap otherwise the studs may eventually buckle due to floor deflections and shortening of the frame.
- Where floor to ceiling heights vary due to field tolerances, it may be necessary to field cut studs to maintain an adequate movement joint. Stud gaps within tracks also must not exceed specified values, otherwise the wall assembly deflects too much.
- Stud perforations must be at least two times the stud width removed from stud ends.
- Double and triple studs at openings must be strapped or tied together to share load.
- The proper installation of bridging including splicing is vital to prevent premature twisting failure of the stud assembly.
- Ties must not be attached near perforations in the webs of studs.
- System performance depends on proper tying together of many components by means of screws or welds. Regarding screws, essential factors are use of specified screw size, adequate tightening while not overturning or allowing the screw to drift, and ensuring screws engage into the stud. Regarding welds, properly qualified welders experienced in welding light gauge steel are required and any welded area must be touched up to restore corrosion protection.
- No materials may be substituted without approval of the designer. Particularly, specified levels of corrosion protection must be supplied and cannot be infringed upon in any installation procedure.

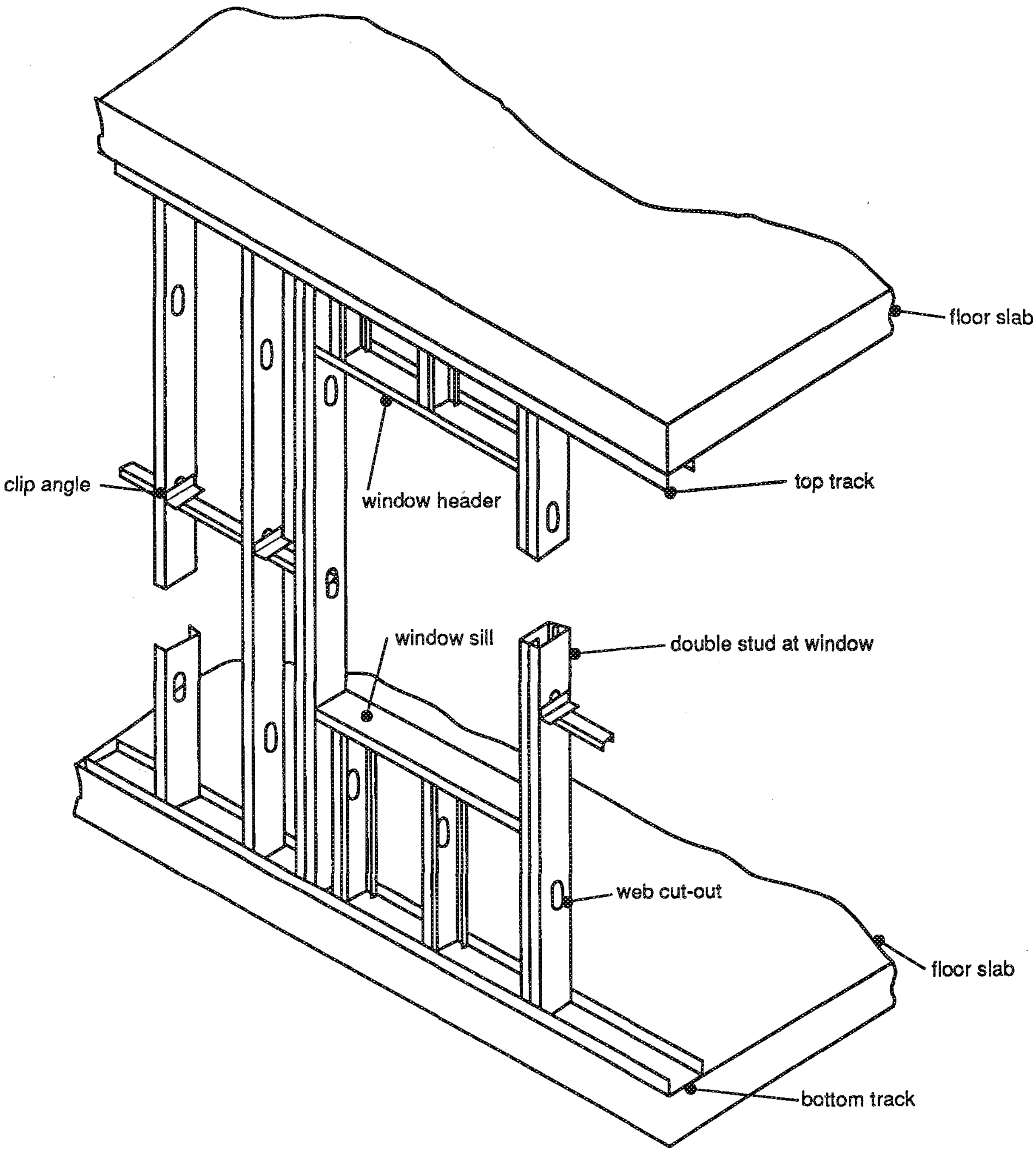



Fig. 8.1 Illustration of components of SS backup walls.

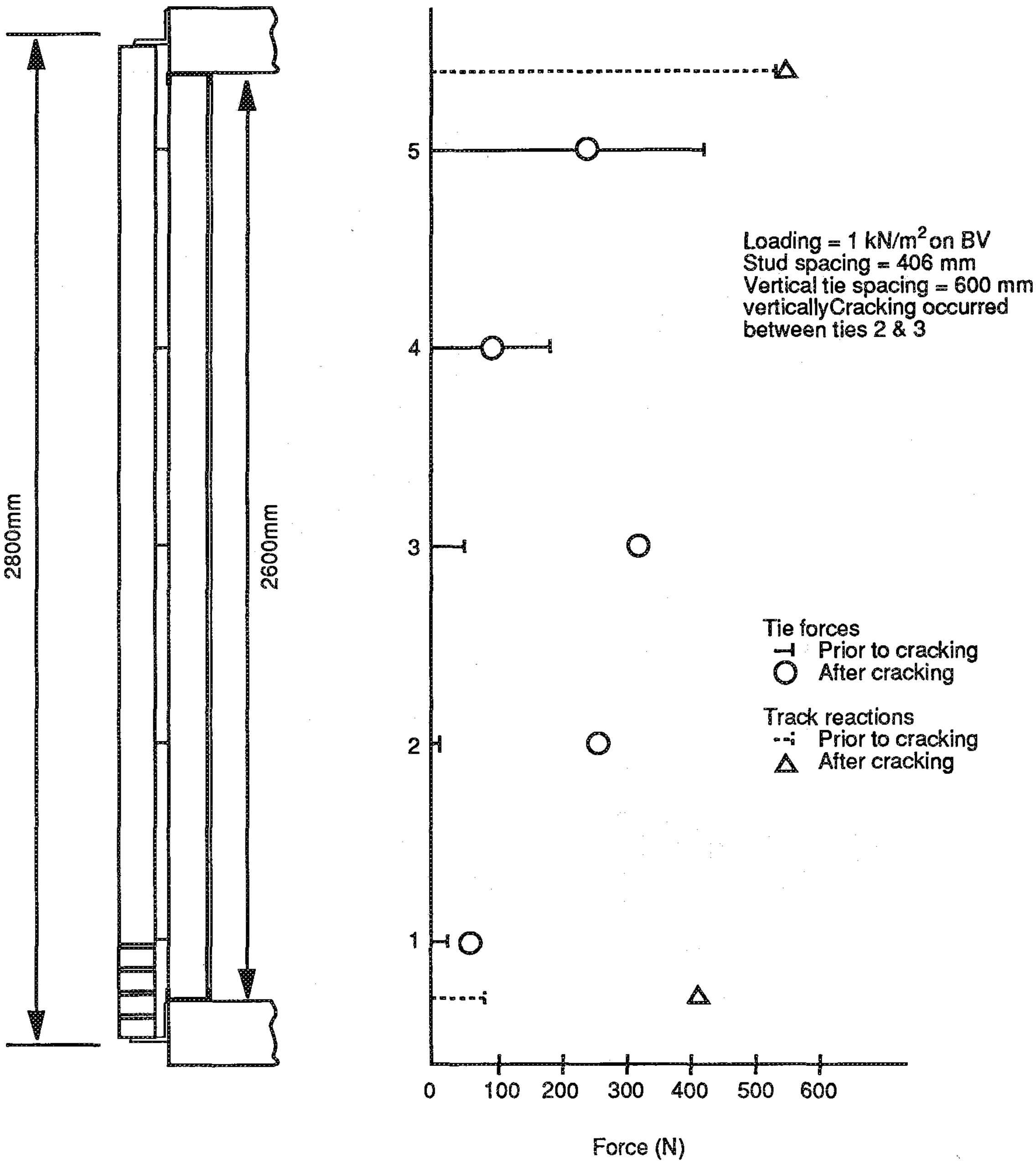



Fig. 8.2 Representative distributions of tie and track forces.

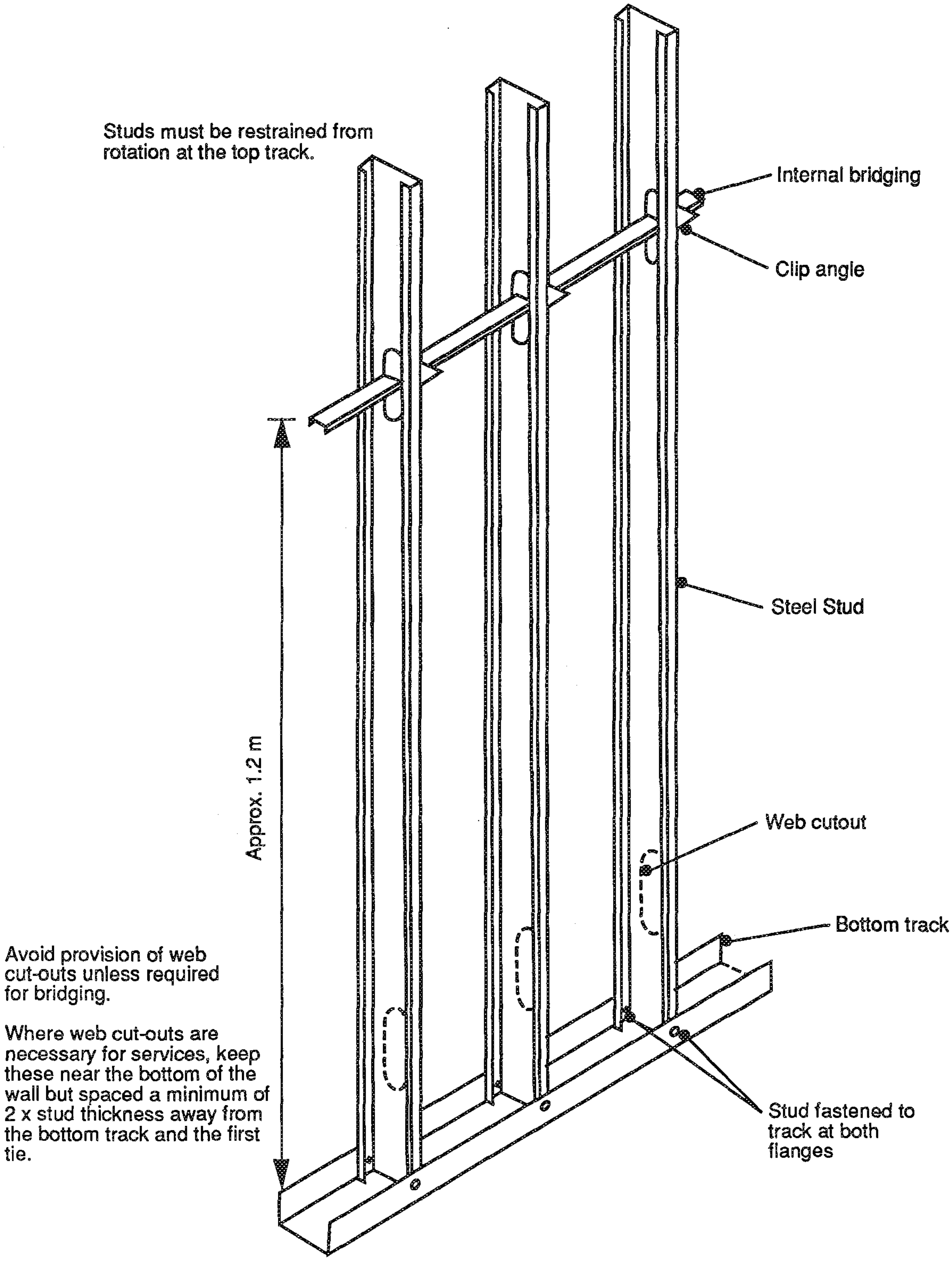
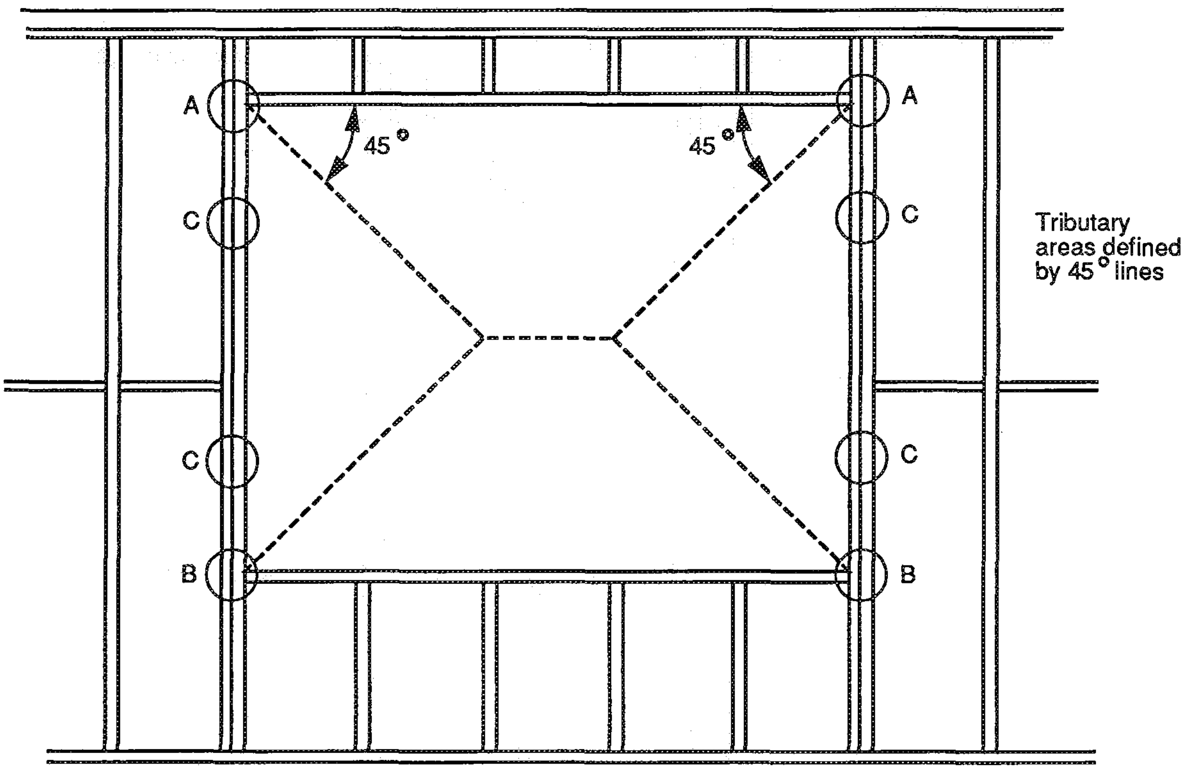



Fig. 8.3 Requirements for discrete bridging or bracing and for web cut-outs




Connection points A will each resist 25% of the wind load for the area above the window.

Connection points B will each resist 25% of the wind load for the area below the window.

If the window is supported along its sodes, approximately 25% of the wind load from the window area will be concentrated at each point C.

If the window is attached top and bottom, points A and B will each carry 25% of the wind load from the window area.

If the window is supported uniformly around its perimeter, the wind load on the window can be distributed according to the tributary areas indicated by the dashed lines.

Fig. 8.4 Load distribution from window areas in walls

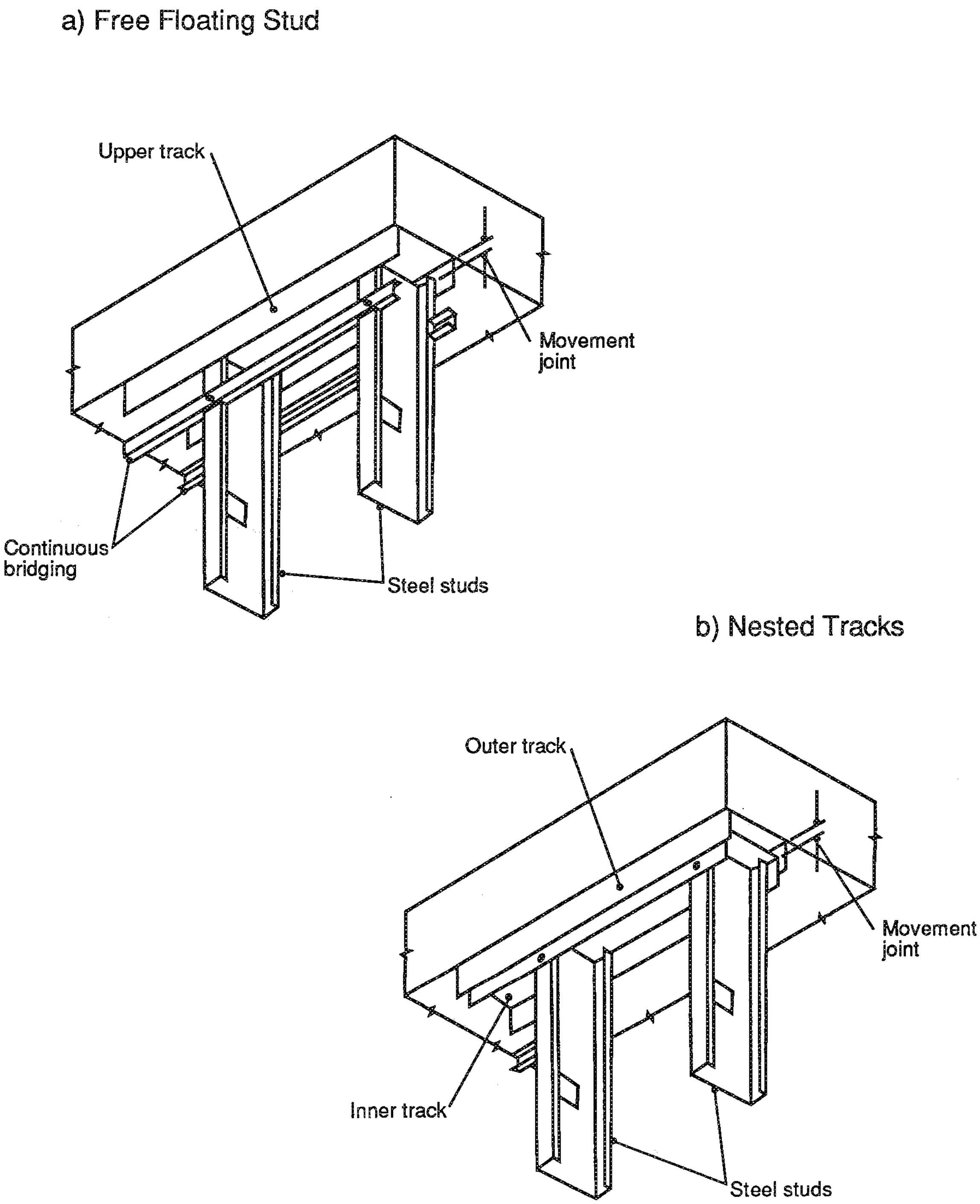
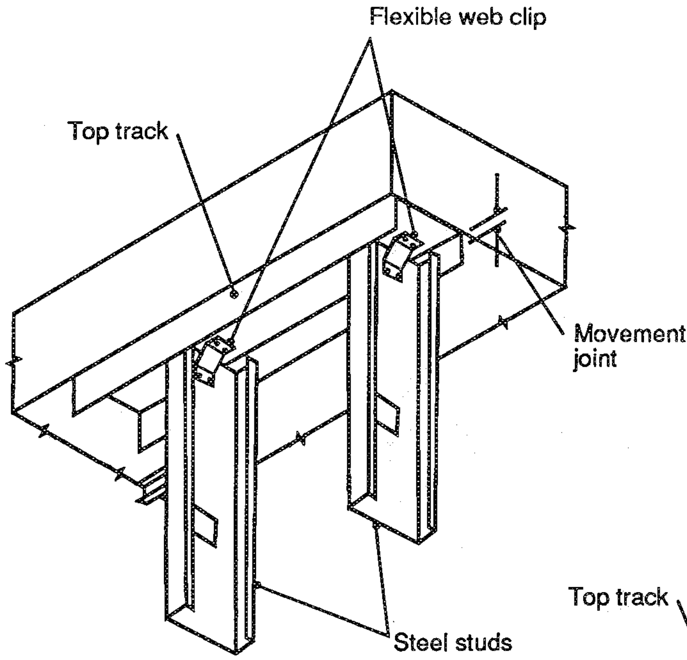
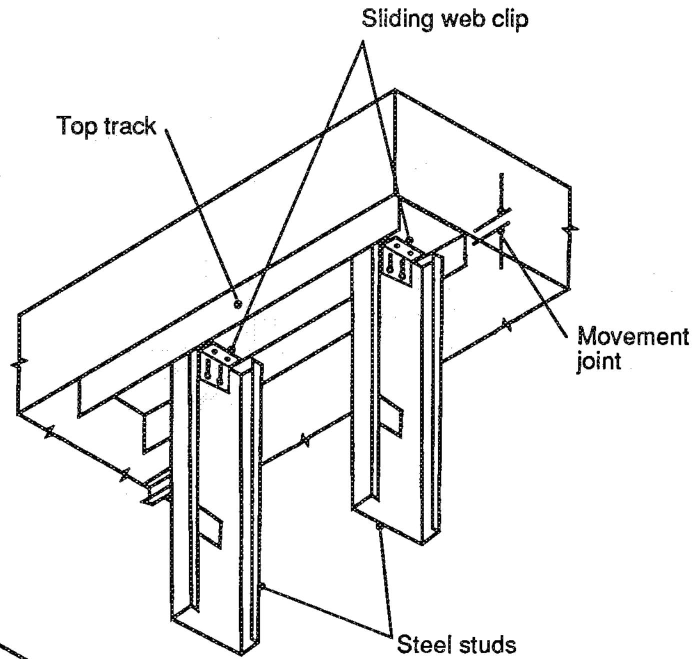


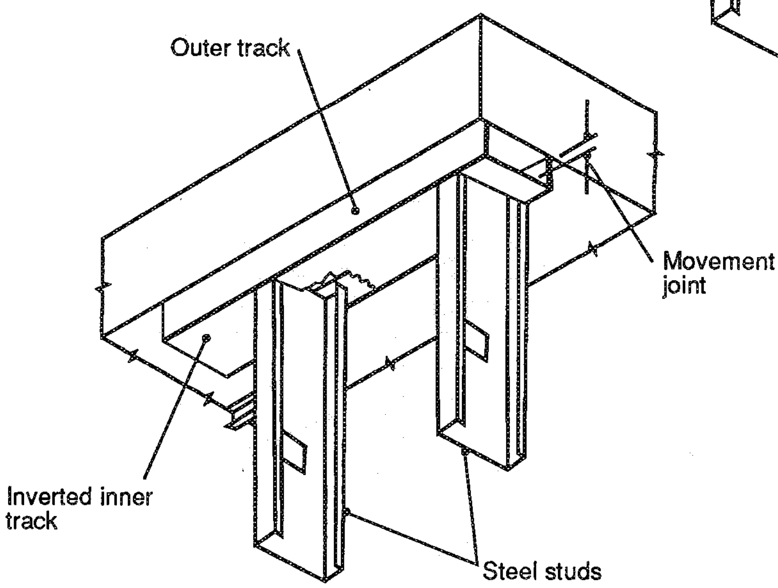

Fig. 8.5 Typical movement joint details for upper track connection




c) Web Connection (Flexible)




d) Web Connection (Sliding)




e) Boxed Track With Free Floating Stud

Fig. 8.5 Continued

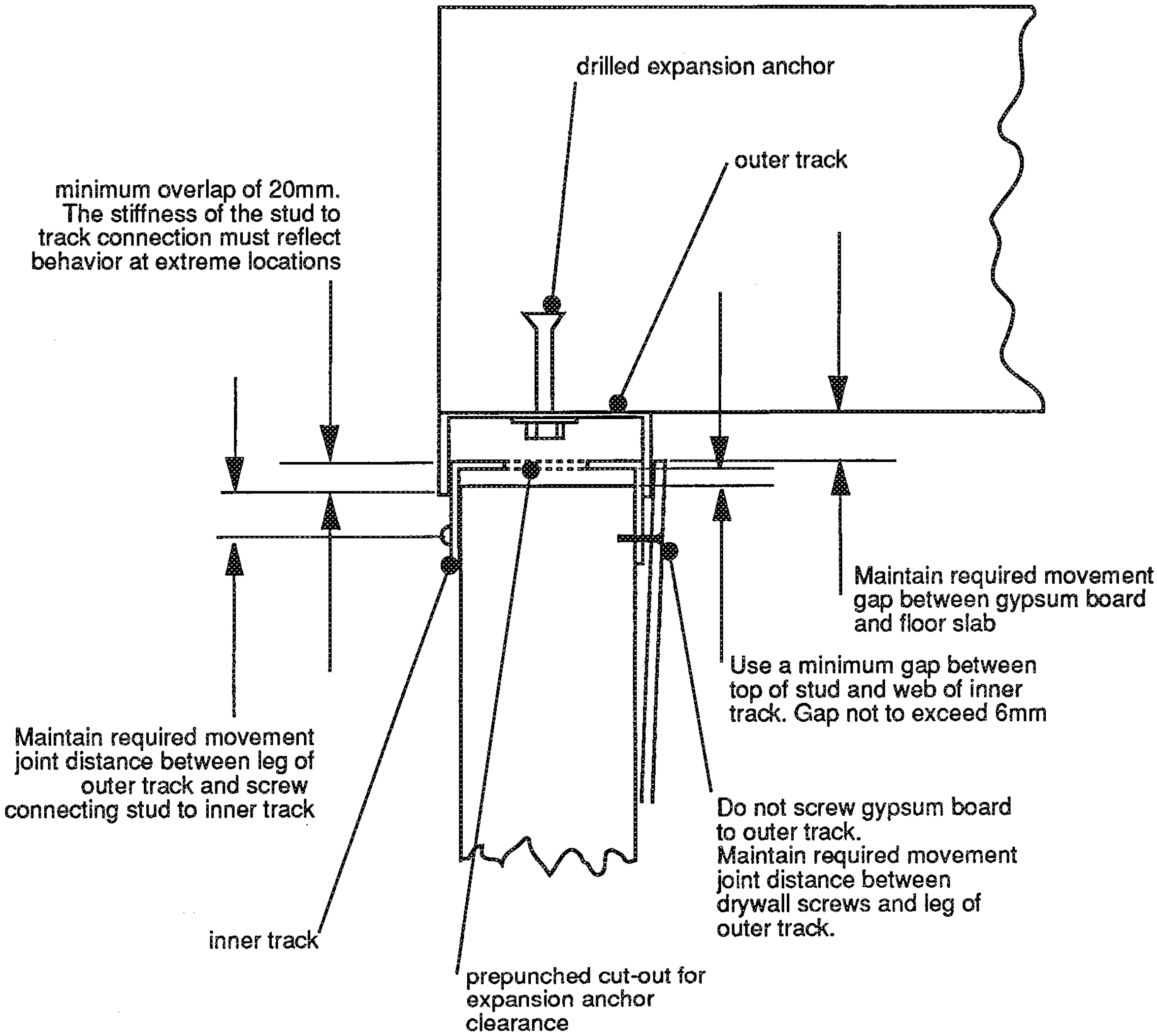



Fig. 8.6 Illustration of requirements to maintain movement capabilities of movement joints.

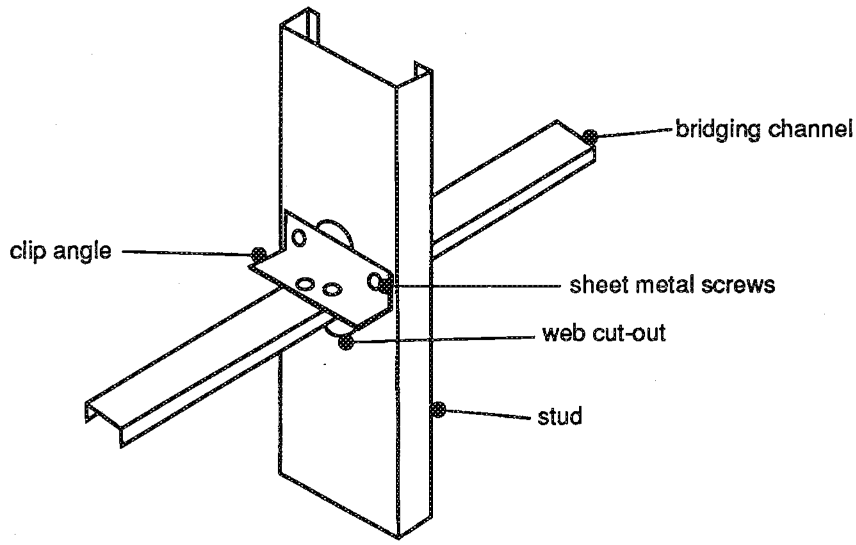



Fig. 8.8 Screwed connection using clip angle to connect internal bridging to stud

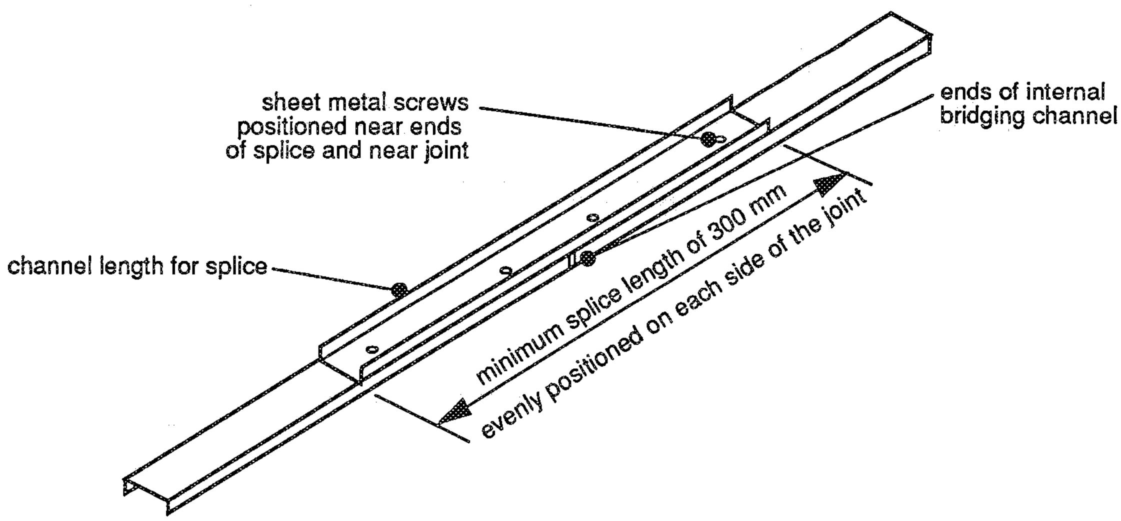



Fig. 8.9 Screwed splice connection for internal bridging

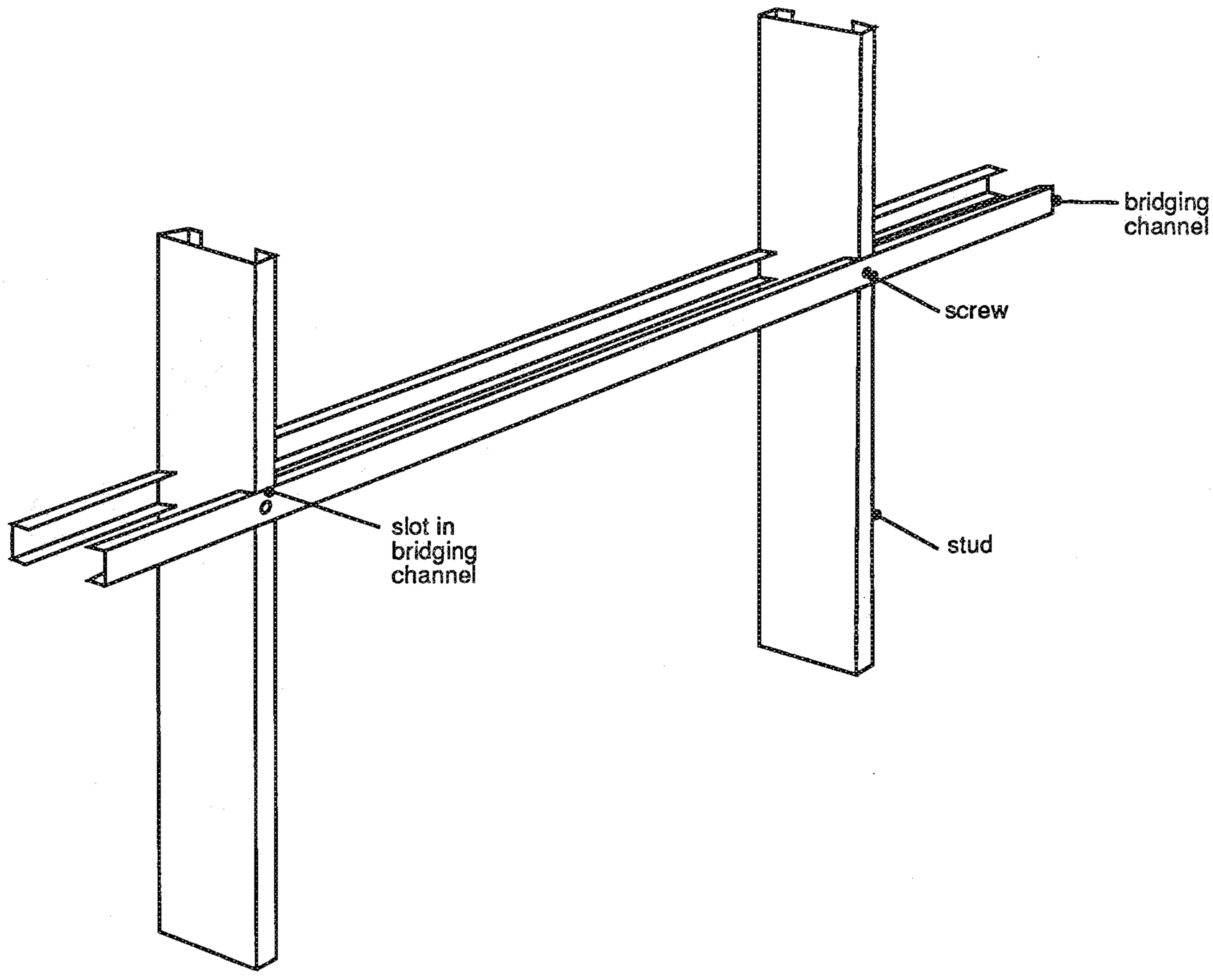
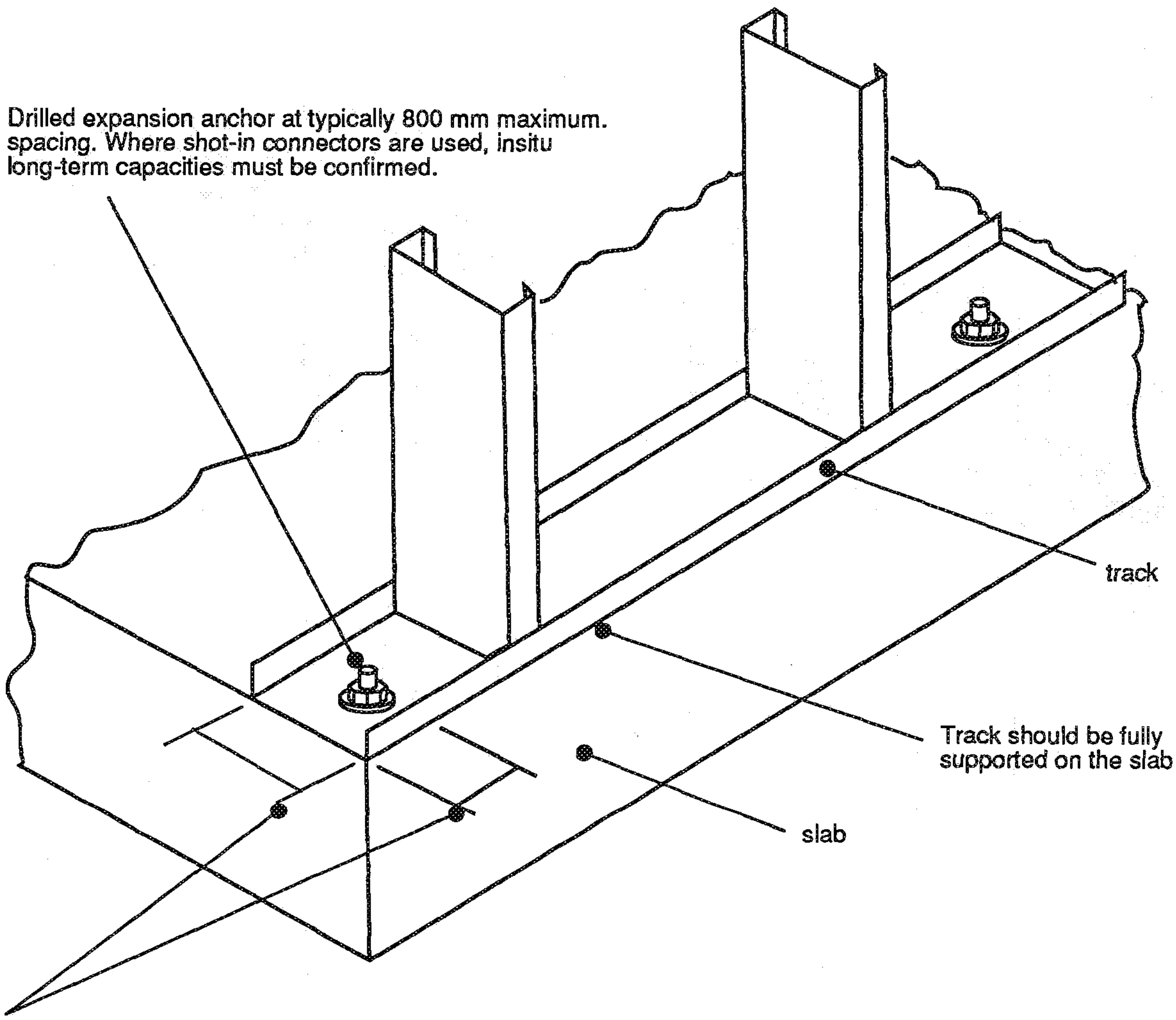



Fig. 8.10 Screwed connection for external bridging




Edge distances for expansion anchors shall conform to manufacturers specifications.

Holes in track shall provide minimum clearance for anchor bolts. Washers must be used and nuts torqued to prevent slipping of track.

Fig. 8.11 Anchor connection between track and slab.

# CHAPTER 9
# GENERAL BUILDING SCIENCE REQUIREMENTS

## 9.1 Introduction

Although the basic functions of components of BV wall systems were described in Chapter 4, general building science design requirements and construction considerations common to BV wall systems will be discussed in the following sections.

## 9.2 Veneer

The veneer wall system design should be based on the idea that waterproofing is a two-stage system with the veneer as the first stage and the cavity-backup wall components providing the second stage. Despite this philosophy, the veneer should be detailed and constructed to provide water and wind resistance so that the effectiveness of the second stage is not continuously tested. If water is allowed to readily penetrate through the veneer, the cavity-backup wall components are really a single stage system which means failure is assured if some fault exists in the system.

While rain may quickly wet exposed areas of the face of a masonry wall, only during the most prolonged or frequent rainfalls are the units and mortar likely to be completely saturated. Usually the materials themselves are too dense for water to pass through quickly. If water passes through, it is generally through fine capillary passages (actually unbonded areas) at the unit to mortar interface, through unfilled mortar joints or through movement joints or cracks. Hence it is important that:

- mortar completely fills head and bed joints
- mortar and bricks are compatible to form good bond to resist cracking
- concave or V-shaped tooling of the exterior face of the mortar joints be done to compact the mortar against the units and help close capillary cracks
- flush, raked or extruded mortar joints which catch water running down the wall and are not compacted, are avoided for rain exposed conditions.

The different types of joints are illustrated in Fig. 9.1.

While leak-proof walls are attainable, the use of a wide variety of materials and the kilometers of mortar joints in any building make it difficult to ensure satisfactory performance. Therefore the open rain screen principle is used where water which penetrates the thickness of the veneer is allowed to run down the inside face and then is directed back to the outside face of the wall through weepholes located at flashing on shelf angles. Figure 9.2 contains illustrations and comments on the functions of the BV wall system components acting as a two stage rain barrier using the rain screen principle.

For the second stage to work most effectively, both pressurization of the cavity and provision of an air tight backing are extremely important. To the extent that the cavity can be pressurized, this will limit the amount of water which is carried through by wind and decrease the tendency for water which penetrates the veneer by capillary action or gravity being carried across the cavity.

There are many reasons why full pressurization of the cavity may not be achieved and as depicted in Figure 9.3 (a), rain may actually be carried in by air passing through weepholes and vents. Movement of air in the cavity can carry the water across to the backup wall and distribute it along the wall area; air leakage through the backup wall can then draw moisture into and through it. Therefore, unless a high level of pressure equalization can be achieved, it is recommended that the size of weepholes and vents be the minimum necessary for removing water from the cavity area. To the extent possible, compartmentalization of cavities should be used to reduce the potential for air pressure differences on the surface of the building causing air flow through cavities. In addition to blocking corners as shown in Figure 9.3(b) vents and weepholes generally should be positioned away from corners where significant large air pressure differences can lead to large air movements through the cavity.

## 9.3 Cavity

The cavity between the veneer and the backup wall is intended to provide a means for water which penetrates through the veneer to be directed back out of the wall using flashing and weep holes at the bottom of the wall.

While many building investigators have reported seeing or testing brick walls which "leaked like a sieve", the fact is that most brick veneer walls show no readily visible signs of leakage. That is, there is no indication of wetting or resulting deterioration either inside or outside the building nor is there any sign that water has ever drained out of the weep holes.

Even where careful construction supervision is employed, some mortar droppings will partially fill the cavity at the bottom of the wall and there will be a tendency for the weep holes to be plugged. In addition, as the brickwork progresses, mortar fins protruding from the back of the wall will tend to bridge across the cavity particularly where the air gap in the cavity is only 25 mm or less in width. These fins plus ties and mortar bridges on ties can all provide paths for water which penetrates the veneer to enter the backup wall rather than be directed back out of the cavity. Figure 9.4 illustrates these difficulties.

In the past, the choice of width of air gap in the cavity has been a design decision. However the adoption from lowrise home building specifications of air gaps as small as 15 mm is not considered to be good practice for walls exposed to wind and rain on highrise structures. Therefore, the recommended air gap in the cavity is 50 mm. This allows some small leeway for correction of line of the building and allows for much improved drainage and air circulation (drying) in the cavity.

While mortar fins are much less likely to bridge across to the backup wall, mortar droppings do tend to fill up the cavity on top of the flashing and plug weepholes. Some possible construction approaches are illustrated in Figure 9.6. However, with excessive amounts of droppings, there is some danger that solid pieces of mortar may build up to a sufficient height to drain water toward the base of the backup wall.

Also worth considering in terms of acceptable rain leakage into the cavity, is the observation that the fewer walls which are put to the ultimate test of extensive wetting, the fewer walls will experience distress due to rain penetration.

## 9.4 Flashing

Although cost varies widely, the selection of flashing material should be based on suitability rather than cost. It is suggested that only superior materials be selected since replacement in the event of failure is extremely expensive.

Flashing must be placed at any interruption in the wall such as at window and door heads, louvers or other wall penetrations, and sills as well as at shelf angle supports for the veneer. It is imperative that the flashing extend through the wall. It must not be stopped behind the face of the wall. Fig. 9.6 illustrates both recommended and deficient flashing installation.

Reglets are often omitted on column faces causing the flashing to be discontinued at columns as shown in Fig. 9.7. Flashing must be comprehensively detailed. To be effective, flashings must be continued across columns and around corners. Due to misalignment of slab edges and the resulting reduction of design cavity width, the flashing may run into interference with the shelf angle anchor bolts as shown in Fig. 9.8. An alternative route of the flashing which can solve the problem is shown in the same figure.

## 9.5 Backup Wall

In addition to structurally supporting the brick veneer, the backup wall provides support for the interior wall facing as well as the vapour, air, and thermal barrier functions.

## 9.6 Ties

Ties provide a potential path for rainwater to cross the cavity to the backup wall. While the drip has been removed for wire ties, use of straight wires is not thought to be a significant source of moisture provided that the ties are placed horizontally as indicated in Figure 9.9. However, where the ties act as a platform for mortar droppings, the larger continuous mortar bridge across the cavity can be a major source of wetting of the backup wall.

## 9.7 Shelf Angle

Aside from the structural functions of shelf angles, there are several areas where building science related functions can be affected. These are:

1. Thermal bridging: Extension of insulation over the upright leg of the angle is feasible and will minimize this.
2. Dimensioning and aesthetic considerations: Since the thickness of the shelf angle does take up part of the height of the veneer, detailing for the overall thickness of the bed joints at the shelf angle must allow for a thin bed of mortar, flashing, the thickness of the shelf angle, and the vertical expansion joint. This has implications regarding the effect of a thicker joint on appearance, and on the overall dimensions and coursing.
3. Bolts: Details must be developed to minimize puncturing of the flashing.
4. Flashing: The flashing must be connected to the backup wall in a manner which will be permanent for the life of the building.

## 9.8 Thermal Insulation

The possible locations for thermal insulation are:

1. In the cavity.
2. In the backup wall.
3. On the interior face of the backup wall.

The general types of insulation are:

1. Rigid insulation.
2. Fibre batt insulation.
3. Loose fill.

Where insulation is placed in the cavity, loose fill insulation is not recommended. Therefore rigid insulation boards are most likely to be used. For this to be effective, the insulation must be held tight to the outside surface of the backup wall. If even a small amount of air is allowed to circulate around the rigid boards, their insulating value is greatly decreased. As a result, the method of installing and securing these insulating boards is very important.

Considerations regarding use of adhesives include:

- difficulty with assuring clean adhering surfaces under on-site conditions
- restrictions against use for exterior applications in cold or wet weather
- need for compatibility of adhesive compound with the insulation

- general stability and long term effectiveness of adhesive including general aging, attack by fungi and micro-organisms, cycling through temperature and humidity cycles, and repeated movements.

Mechanical connection using the ties as mounting devices or directly attaching the insulation to the backup wall by means of a screw and washer assembly are likely more reliable in the long term.

For current construction methods, use of insulation in the cavity requires that the wall be built in two stages with the backup wall built first. The advantage of keeping the backup wall warm and reducing the effects of thermal bridges leads to the recommendation of placement of insulation in the cavity.

For SS backup walls, insulation in the wall is usually a "friction fit" thermal batt where the width of the batt is slightly larger than the stud spacing. Stud spacing must be carefully controlled so that friction is effective in keeping the batts in place. For this situation the batt must fill the entire intra-stud space so that air circulation is restricted.

Use of insulation on the inside of the backup wall is normally limited to CM backup. It has the advantage of easy installation and inspection conditions. However the thermal bridging through the floor and the cycling of the backup wall through the entire temperature range are substantial disadvantages.

It is important not to leave any gap between the insulation and the floor or ceiling. In case of hung ceilings or ceilings attached to the bottom chord of open-web joists, the insulation should be continued above the ceiling to the bottom of the structural slab as shown in Fig. 9.10. For effective control of air and vapour leakage, air and vapour barriers should also be continued as illustrated. Besides, if the air barrier is not continued, the insulation may be separated from the backup by the pressure of infiltrating air as shown in Fig. 9.11. As illustrated in Figure 9.12, proper butting of the edges of the insulation and attention to details at the air barrier can help minimize direct heat loss and other detrimental affects due to air exfiltration.

## 9.9 Air and Vapour Barriers

There has been some misunderstanding about the independent functions of air barriers and vapour barriers in that vapour barriers are intended to control transmission of water vapour through materials whereas air barriers are to limit the flow of air through the wall. The cause of this misunderstanding may be due to the fact that it is very difficult to assure that either barrier performs only its single function. For example, polyethylene film is commonly used as a vapour barrier, yet will also act to resist the flow of air. Alternatively most types of sheathing used as air barriers tend to restrict the transmission of water vapour. Therefore a common problem may be that many wall assemblies contain essentially a double set of barriers.

If it is assumed that some degree of imperfection exists in the construction of the wall components - as inevitably will occur on site - then the design should be based on evaluation of the potential vulnerability of the effects of these imperfections. In practice this means that moisture may become trapped between barriers and above all that the degree of wetness and duration of wetness at certain critical locations may render designs vulnerable to premature deterioration and distress. That is, even though in theory these designs would function satisfactorily for conditions of perfect construction, the real world may introduce significantly reduced service lives. Of major concern to designers is the potential corrosion of steel components, weakening or deterioration of the sheathing material and wetting of the insulation which will decrease its effectiveness and lengthen time of wetness of steel components. Growth of mold or fungus in the wall may also be an undesirable effect of trapping moisture in the wall.

Tests, analyses, 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. In general the design of the air barrier/vapour barrier system should aim to minimize gross errors and to tolerate minor errors. Assessment of the suitability of the design for long term performance should account for the fact that these barriers do not remain accessible to inspection or repair.

## 9.10 Sealing

To provide effective air and vapour barriers, it is usually necessary to seal joints in the barrier materials and to provide continuity around the edges and at openings such as windows, doors, or access for services. Normally either caulking or taping will be specified. However, in addition the joint should be fully detailed and dimensioned with due consideration of the need to accommodate large variations in joint dimensions, building deflections and thermal movements.

If the sealant material is to become built in to the wall and thus hidden from later inspection and maintenance, it must have qualities which guarantee satisfactory performance over the life of the building, even when installation or service conditions are less than perfect.

### 9.11 Sealant, Backing and Filler

Movement joints might perform their intended function best without any filler material in the joint. However a seal is required to prevent the passage of liquids or gases through the joints while not restricting the differential movement between the components being joined. It is the sealant which is the primary material that forms the seal; a backing and often a filler are used in deep movement joints particularly in masonry walls to facilitate the formation of a proper seal. The sealing components of a typical movement joint are shown in Fig. 9.13.

Figure 9.14 contains sketches of a variety of sealant shapes and bond breakers where the shape of the sealant bead has a considerable effect on its durability and performance. Sealing compound simply smeared over a crack is sure to fail because of the exposure of the material and the high concentration of stress at the crack (Figs. 9.14 (a)).

It is important to note that filler material such as asphalt impregnated fibreboard, rigid polystyrene or other plastics, or solid rubber are capable of transmitting sizeable compressive forces across the movement joints. Since transmission of forces across either vertical or horizontal movement joints negates the intent, these material should not be used. An alternative to using a filler material is to use a second set of backer rod and sealant to provide improved air and water tightness.

### 9.12 Vulnerabilities

It is recommended that every design should be subjected to a "vulnerability assessment" before a final decision is made. The vulnerability assessment should include:

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

### 9.13 Construction

The complexity or "fussiness" of the wall details can have a significant effect on the frequency of imperfections where, as a general rule, the best details are the simplest and are those which are not highly sensitive to the "weakest link" for successful performance. For example, is successful performance heavily dependent on the long term performance of a single line of caulking located in a difficult area of access which is later covered over? Similarly, the level of workmanship required particularly as it relates to special skills or special training of the workmen can adversely affect the effectiveness of a design. In general, it is desirable to minimize the number of trades involved in the construction of a wall.

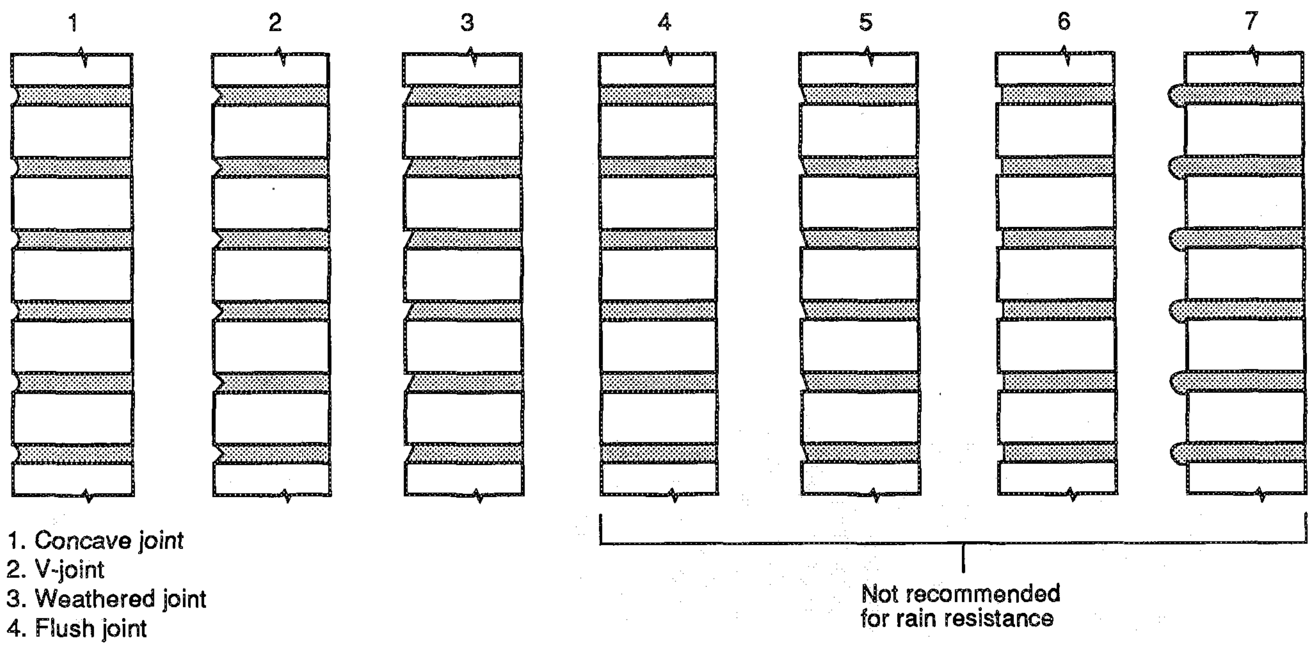
Finally, the degree to which inspection can be effective is very important. If an aspect of construction requires essentially continuous inspection because key features quickly become hidden behind other layers of construction then this should be considered to be a weakness in the design. Unless use of the materials and the construction procedure are basically errorproof, a greater likelihood of construction deficiencies must be anticipated. In much the same way, the ability to correct faults and to carry out regular maintenance should be among the foremost considerations when assessing the acceptability of a particular design. While this is fundamental to many of the things we build, for some reason it does not seem to have been identified as a key feature for air barrier and vapour barrier construction.

### 9.14 Key Construction Requirements

From the preceding discussion of building science requirements and additional considerations, the following key requirements for construction stand out:

- Good quality workmanship must be used in construction of the veneer so that full mortar joints tooled on the exterior face will be resistant to rain penetration.
- Mortar droppings must be minimized and weepholes must not be plugged in order to ensure that moisture will drain from the cavity.
- Undamaged flashing must be securely anchored to the backup wall with properly lapped joints to achieve a continuous membrane to collect and direct water out of the cavity.
- Flashing must be sufficiently wide to reach the exterior face of the veneer in order that water from the cavity is returned all the way to the exterior.

- At shelf angle locations, proper installation of caulking on backer rod is required to prevent water entering the top of the veneer and the cavity.
- Installation of ties must not provide a direct path to carry water to the backup wall and ties perforating exterior components of the backup wall must be sealed as specified.
- Shelf angles must not be tilted backward, otherwise water will accumulate rather than being directed out of the wall.
- Regarding installation of thermal insulation, gaps between insulation and other wall components and open joints between units of insulation must be avoided. Otherwise circulation of air can drastically reduce the benefit of insulation.
- Where insulation, air and vapour barriers are hidden from view by subsequent construction, special care is required to assure that flaws which could allow exfiltration of moist air are repaired. Continuity must be maintained across all joints in these materials and at intersections with other building elements such as slabs, windows and columns.
- Manufacturers' instructions for use of the specified sealant must be closely followed and only specified filler material should be used to avoid restriction of movement and distress due to buildup of stress.
- No materials may be substituted without the approval of the designer.
- Work in progress must be adequately protected from damage due to weather or construction activities by other trades.




1. Concave joint
2. V-joint
3. Weathered joint
4. Flush joint
5. Struck joint
6. Raked joint
7. Extruded

Fig. 9.1 Illustration of different types of mortar joints.

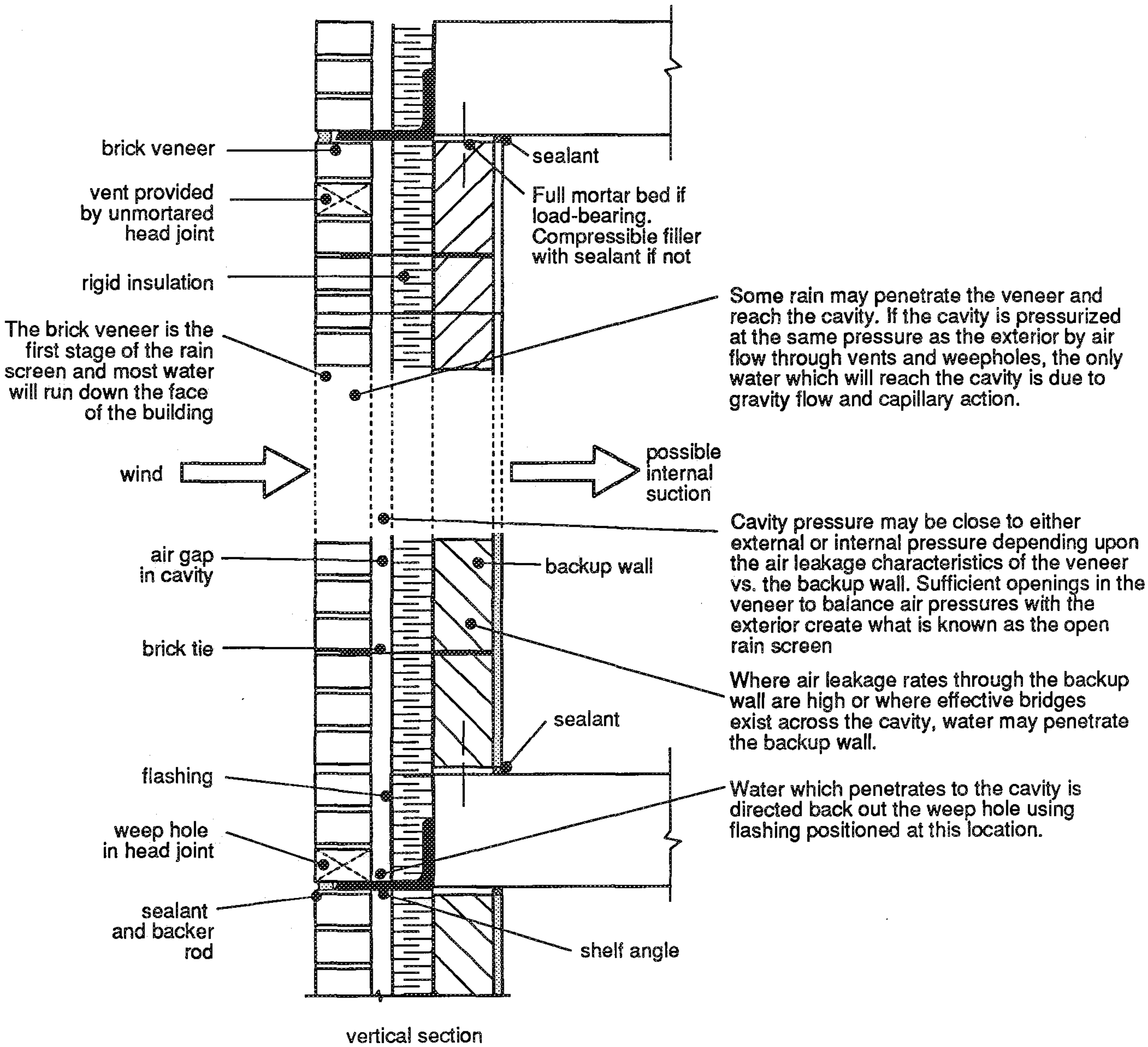



Fig. 9.2 Use of brick veneer as a rain screen.

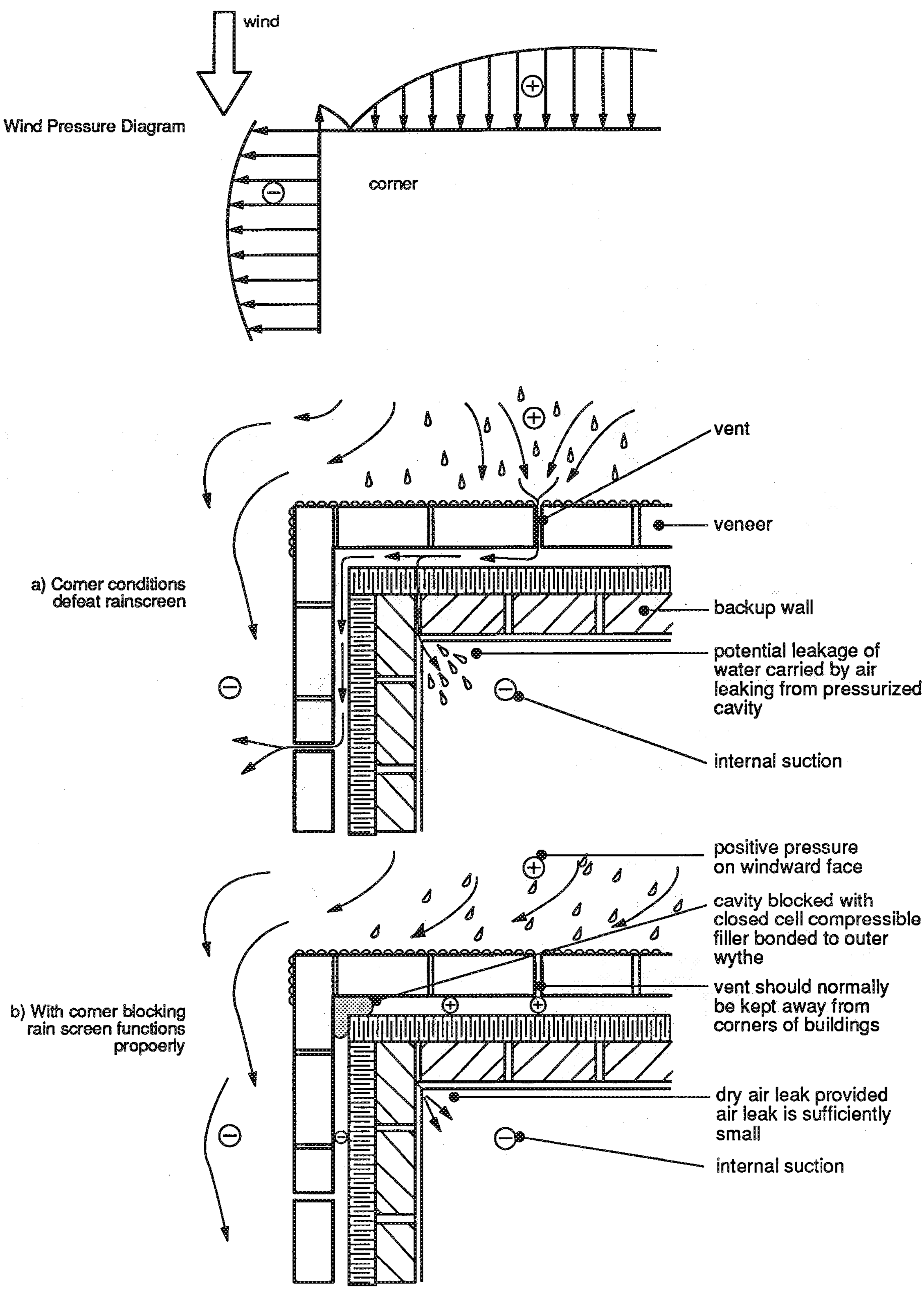



Fig. 9.3 Illustration of pressure equalization problems at corners of buildings.

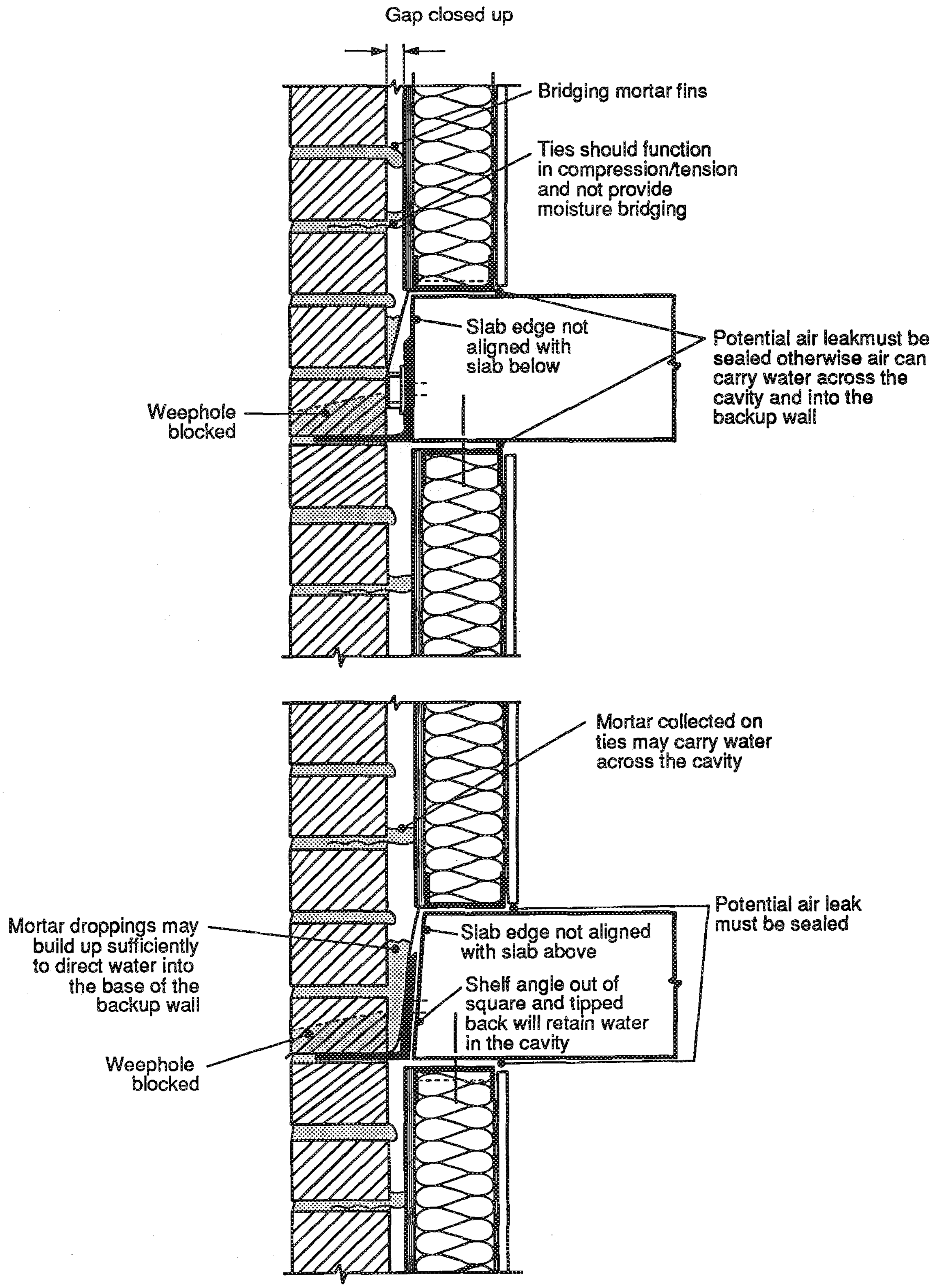
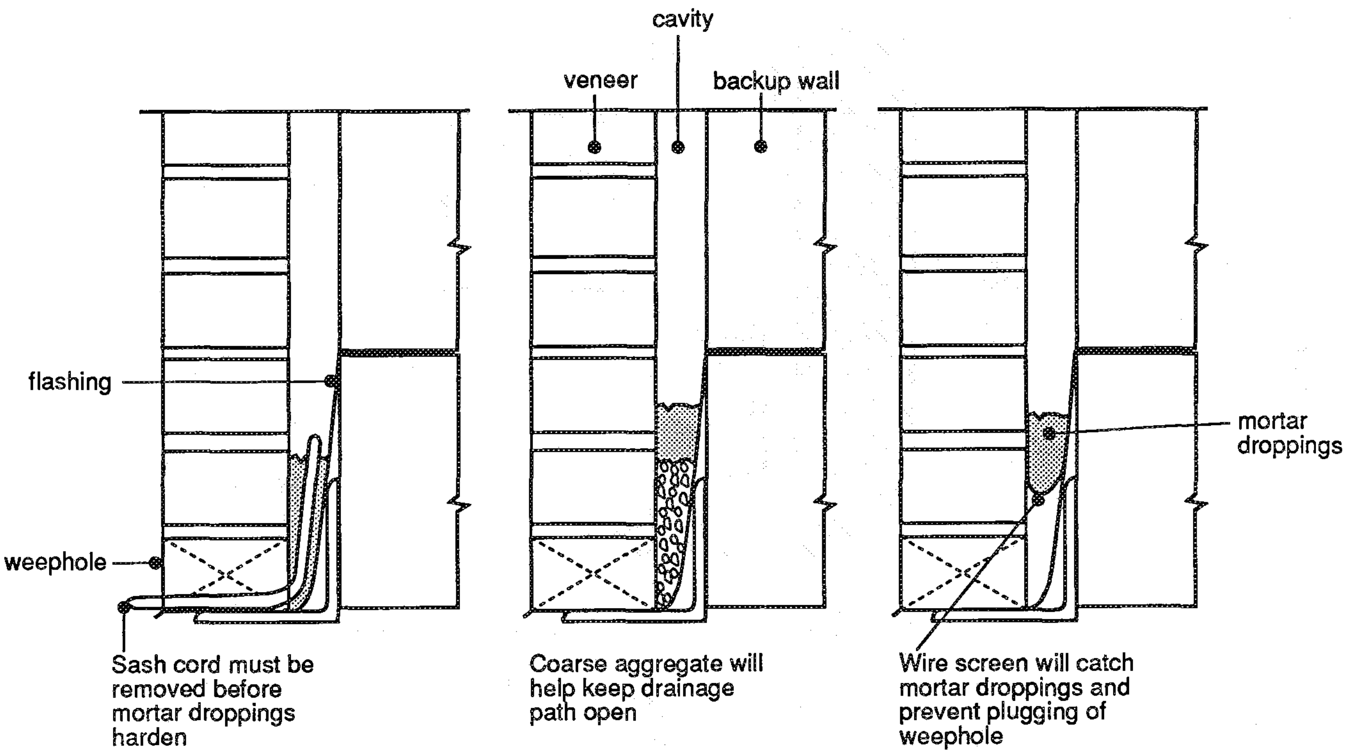



Fig. 9.4 Mechanisms for transportation of water across the cavity.




The most effective method of preventing plugging of weepholes is to minimize mortar droppings. Preventative measures include:

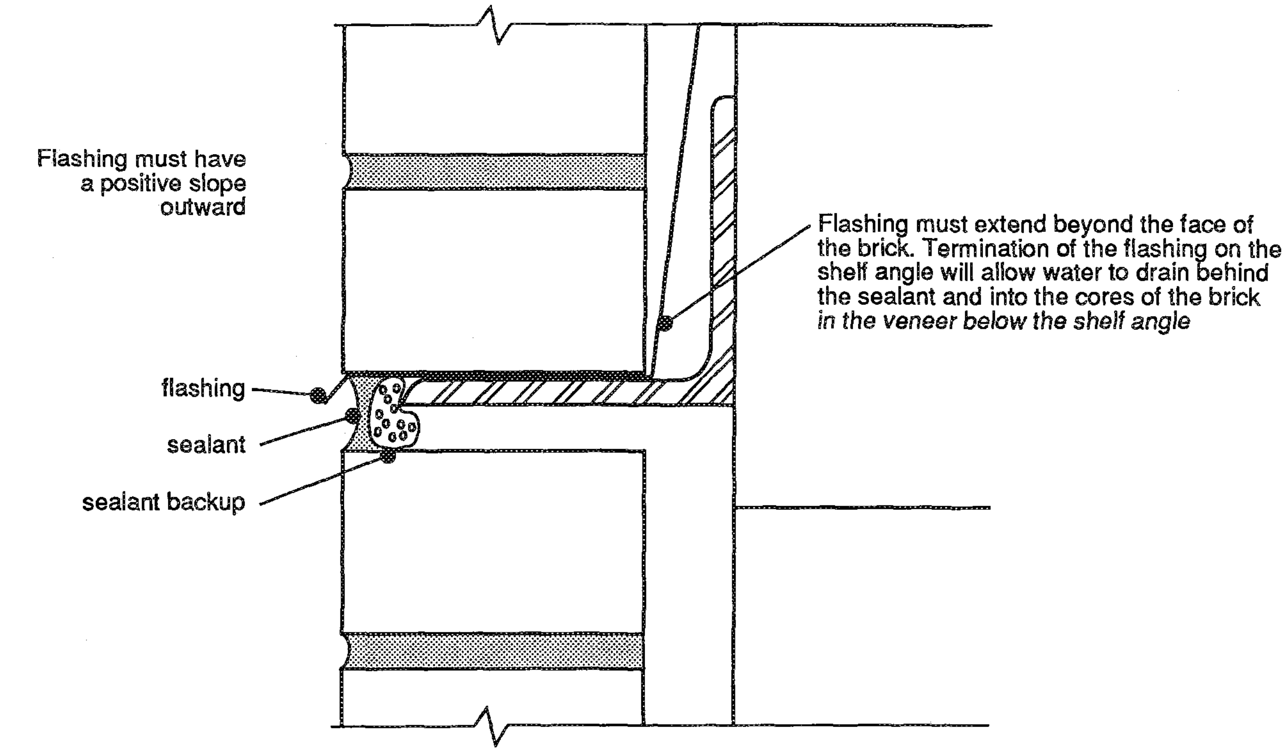
Bevelling back mortar will reduce protrusion of mortar fins into cavity

A narrow strip of wood supported by wires can be lifted up the cavity as construction progresses. This will help catch mortar droppings

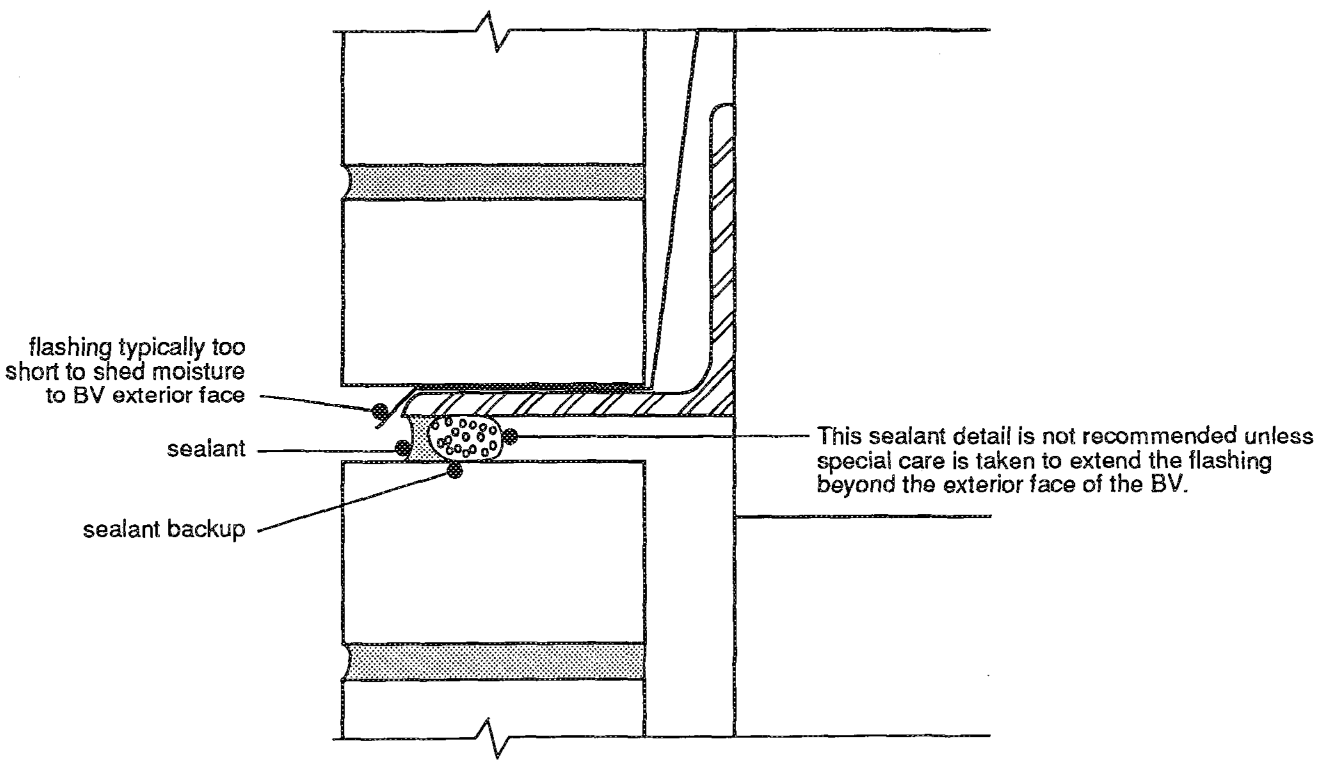
Rope laid at the base of the cavity and raised after the mortar droppings have partially hardened will help create a drainage path and prevent the mortar from becoming a solid mass.

Bricks left out of the bottom course will allow the cavity to be cleaned out each day.

Fig. 9.5 Methods of assisting drainage through weepholes.




a) Recommended flashing and sealant detail




b) Deficient flashing and sealant detail

Fig. 9.6 Flashing and sealant detail at a horizontal shelf angle joint.

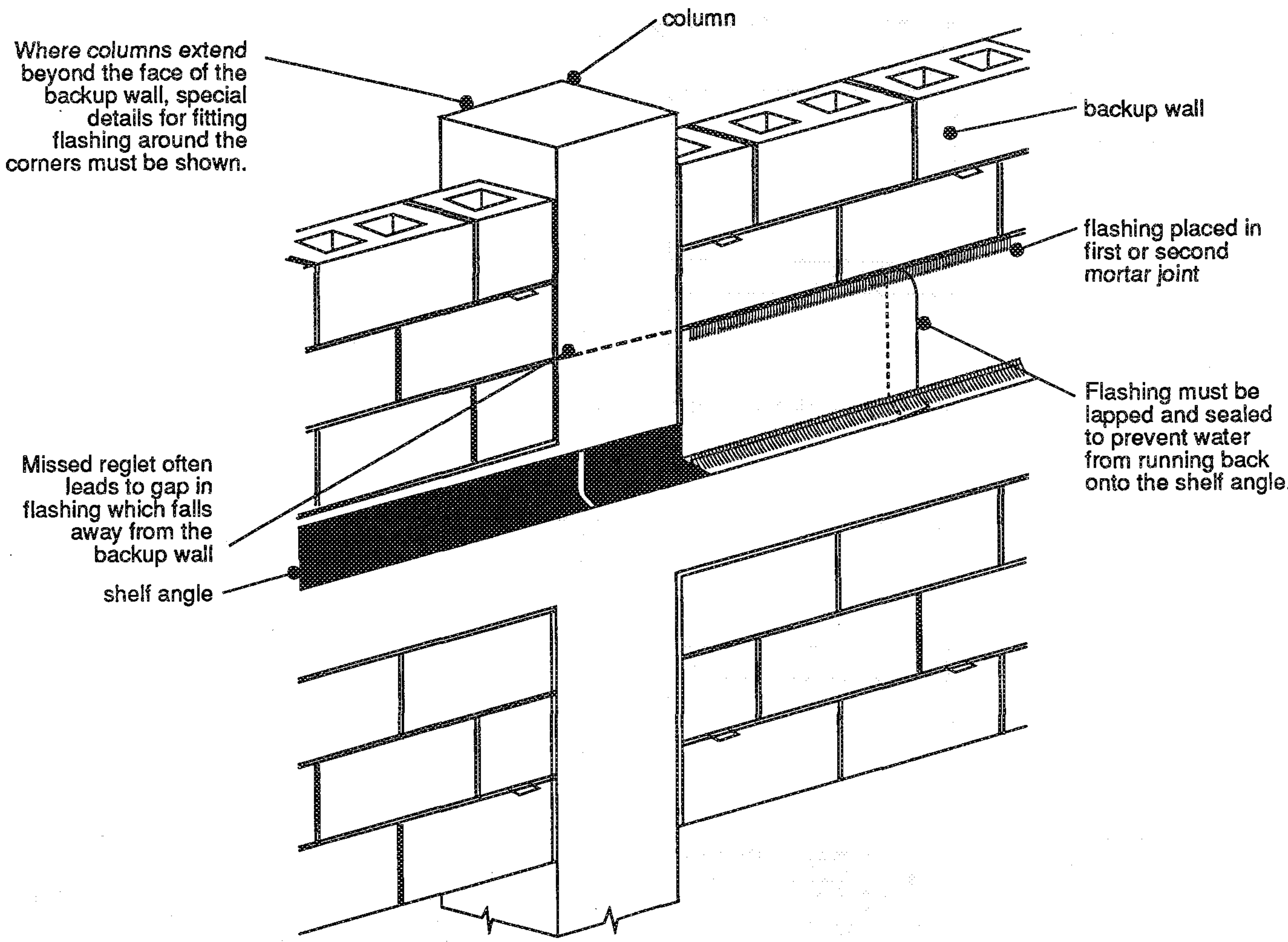



Fig. 9.7 Detailing of flashing to maintain continuity.

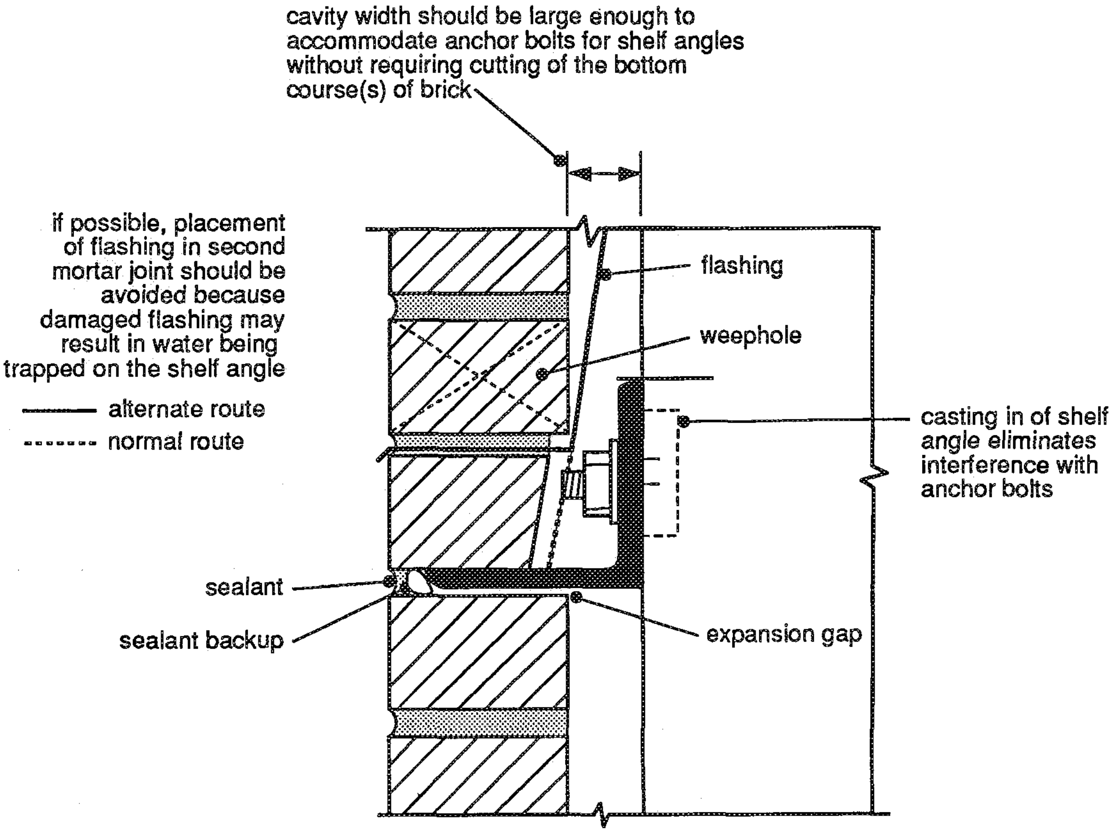



Fig. 9.8 Flashing raised above shelf angle to avoid interference with bolt.

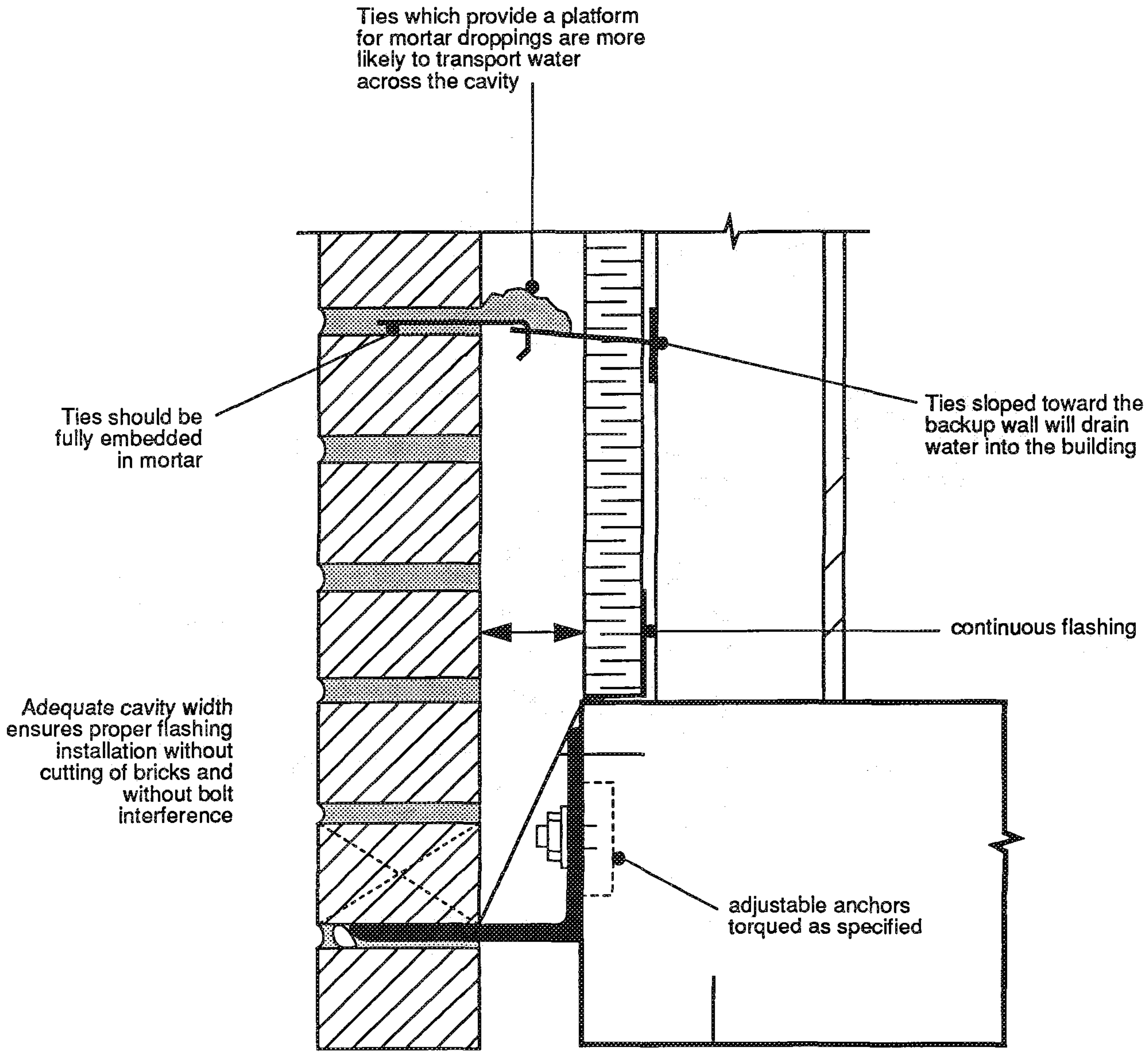



Fig. 9.9 Potential paths for moisture penetration associated with ties.

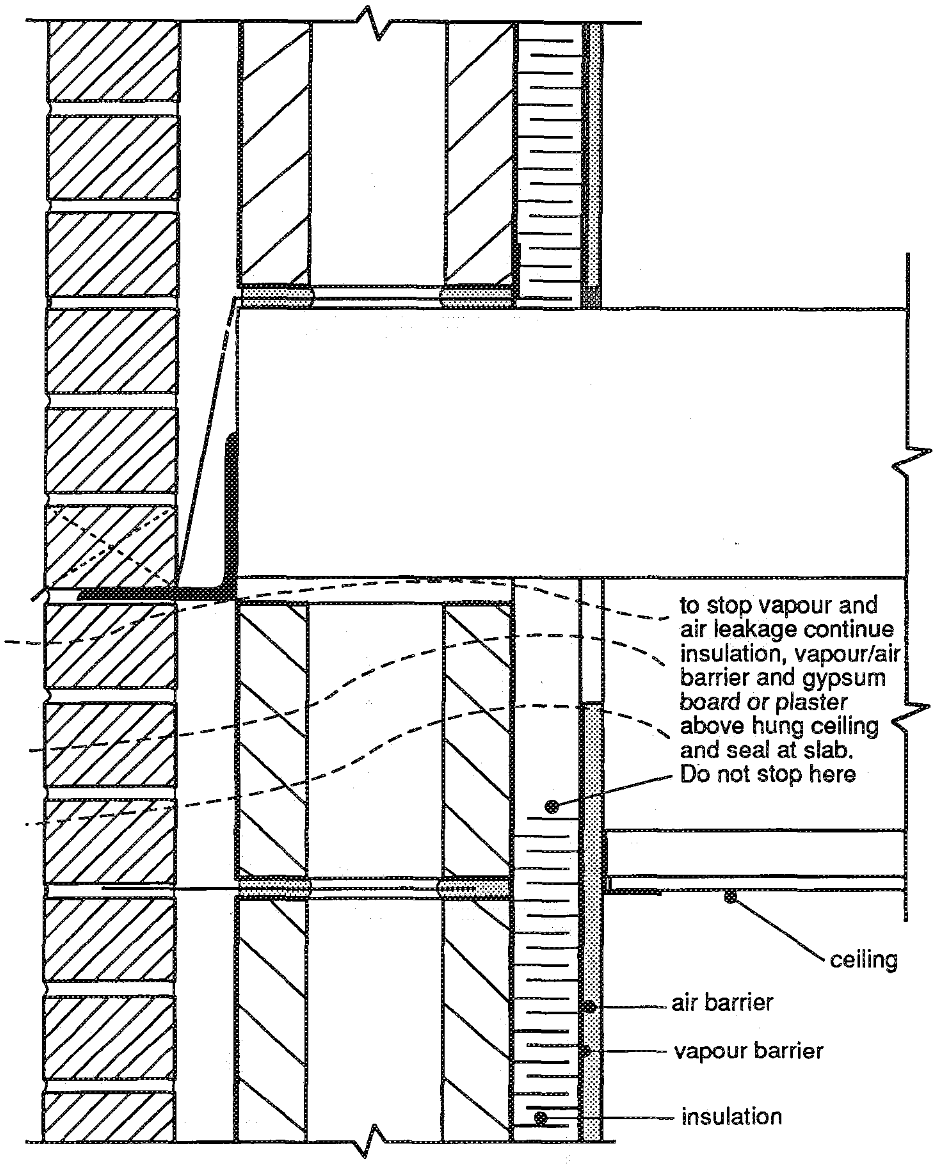



Fig. 9.10 Illustration of the need to continue insulation, air and vapour barriers above hung ceiling.

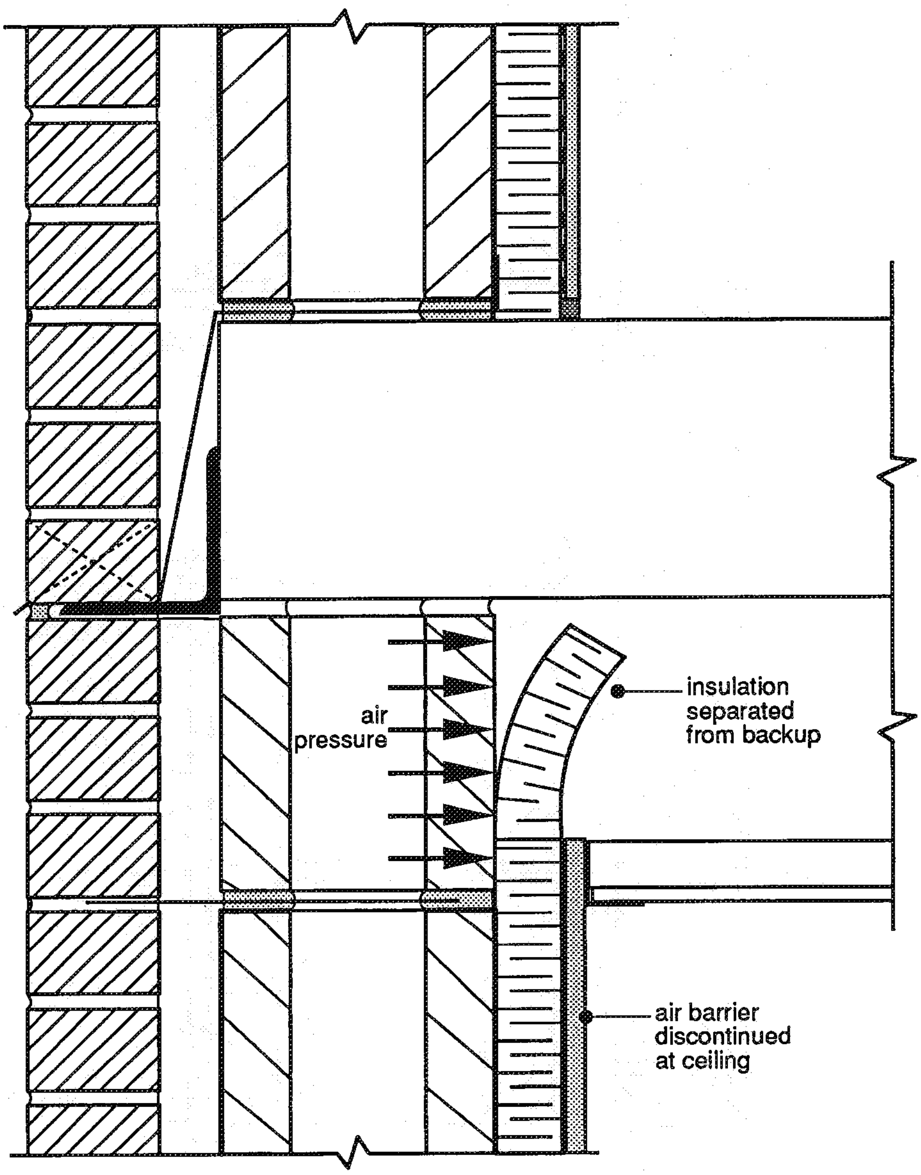



Fig. 9.11 Illustration of air pressure acting on unsupported insulation.

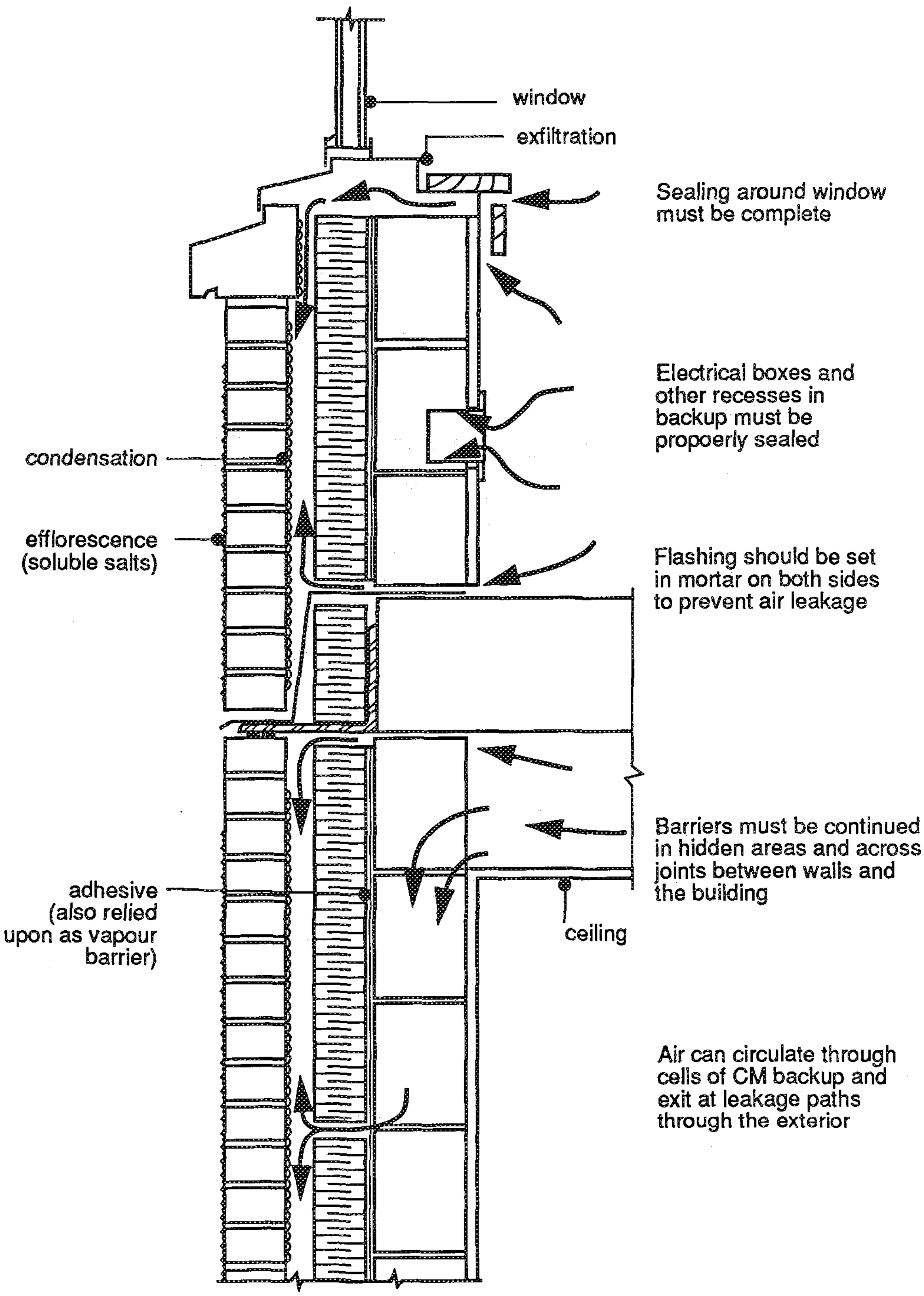



Fig. 9.12 Example of sources of exfiltrating air which may result in condensation of air borne moisture.

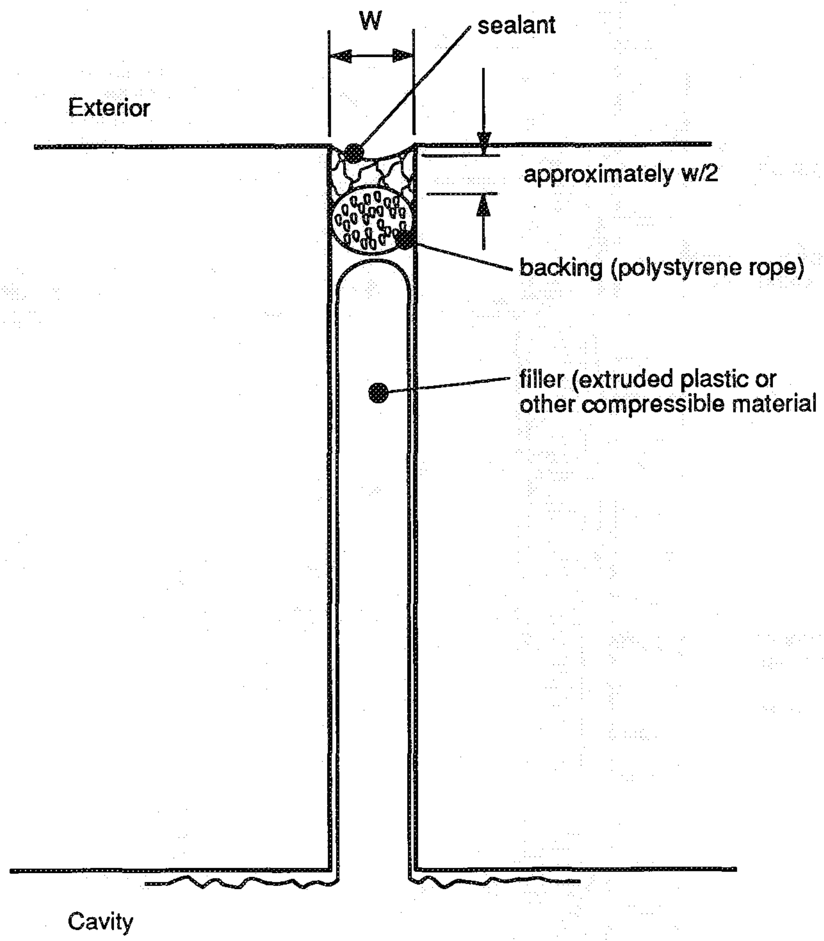



Fig. 9.13 Components of a joint seal.

# CHAPTER 10
# BV/CM SYSTEM: BUILDING SCIENCE REQUIREMENTS

## 10.1 Provision of a Weather Screen

As discussed in Chapter 9, properly built brick veneer will be resistant to water penetration with only small leakage paths existing. Also subsequent cracking of the veneer should not be encountered where:

- the stiff backup limits out-of-plane deflection due to wind or seismic forces
- adequate size and spacing of control joints are provided
- deflections of the supporting structure are limited.

The flashing must satisfy the conditions outlined in Chapter 9, which include provision of sufficiently strong flashing to resist tearing in the wind prior to construction of the veneer. In addition, the flashing must be securely fastened to the backup wall. In this regard, the following comments are provided:

- The flashing should extend up the backup wall 190 mm if on the outside of the wall or 100 mm if on the inside of the wall
- Where the flashing is on the outside of the block wall, attachment by adhesives or mechanical fasteners alone is not recommended because of the possibility of water running down between the block and the flashing
- For flashing on the outside of the wall it is recommended that the flashing be bent into and anchored a minimum 15 mm in the mortar joint immediately above the first course of blocks as illustrated in Fig. 10.1. The mortar joint may be raked out to allow the flashing to be anchored in place at a later stage of construction. If the wall is not parged the flashing should be placed outward of the insulation up to the point where it is bent horizontally into the mortar joint. If parging is used, the flashing may be anchored beneath the first block.
- Where insulation and/or an interior finishing layer are applied to the inside of the CM backup wall, the flashing may extend through the base of the wall and be fastened a minimum of 100 mm up the inside face of the block.

## 10.2 Resistance to Heat Flow

The main insulation will either be rigid insulation on the exterior face of the backup wall or either rigid or batt insulation on the interior. As discussed in Section 9.8, for the insulation to be effective it is very important that it be held tight to either the inside or outside face of the backup wall. Depending on how smooth and flat the block wall is, it may be necessary to parge the surface to provide a smooth flat surface. Alternately air circulation behind rigid cellular plastic insulation may be sufficiently restricted if a full bed or grid of adhesive is used in addition to mechanical anchorage. Rigid fibrous insulating boards may be sufficiently flexible that mechanical anchors can pull them into firm contact with the backup wall.

## 10.3 Resistance to Vapour Transmission

For CM backup walls, there are four possible locations for the vapour barrier as illustrated in Fig. 10.2.

- At the interior drywall or other interior layer of finishing: This is necessary where insulation is placed on the inside of the backup wall. Depending on moisture conditions, the vapour barrier can take the form of
  - a painted surface on the gypsum board consisting of 2 coats of oil based or alkyd paint
  - a 0.15 mm thick polyethylene sheet over the insulation
  - the insulation itself if it is sufficiently impermeable.
- On the inside face of the block wall: Where the block wall is exposed, painting the wall or otherwise sealing it with materials which have high resistance to water vapour transmission can be effective.
- Sheets on the external face of the CM: Where the insulation is placed on the outside face of the backup wall, the vapour barrier may take the form of a self-adhesive, torched on, or sprayed on impermeable sheet applied directly to the block. This layer must be continuous, able to accommodate moving cracks, and over the long term remain firmly attached despite air and vapour pressure and aging effects.
- Rigid insulation attached to the external face of the CM: Rigid insulation which has adequate resistance to water vapour transmission can be the vapour barrier provided that it is held firmly in contact with the wall. In some cases the mastic used to hold the insulation will itself form the vapour barrier if the wall is covered with a full bed of adhesive. Alternately, the joints between the rigid insulation boards must be fully buttered with adhesive and provision must be made to accommodate movement around the edges of the panel.

## 10.4 Resistance to Air Flow

While block walls may appear to be a solid airtight form of construction, they in fact have characteristically high air leakage rates. Fortunately CM backup walls are amenable to practicable and inspectable air tightening measures which in general should be located inward of the dewpoint. The most common methods of creating airtight block walls are as follows:

- Parging of the exterior or interior of the block wall is effective in greatly reducing air leakage.
- For an improved quality of air barrier using parging, the air barrier and vapour barrier may be considered as a combined system where the parging provides an excellent base for application of the vapour barrier material.
- The interior gypsum board, or other wall covering, which may be used to provide the vapour barrier can also be used as the air barrier following the same design and construction considerations outlined in Section 9.9.
- If interior or exterior rigid insulation is used as a vapour barrier, it may also be used to form part of the air barrier system where adhesion to the backup wall, fully buttered joints, mechanical anchorage to the backup wall, and provisions to span the movement joint provide a structurally sound and continuous barrier.

## 10.5 Location of Wall Components

For highrise construction, locating thermal insulation, air barrier, and vapour barrier on the interior face of the CM backup wall provides good construction access and is amenable to ready inspection. Also correction of construction deficiencies as well as eventual maintenance and repair measures are much more feasible than for designs where these wall components are located in the cavity. In addition, these barrier wall components can be installed by tradesmen more experienced and knowledgeable in these areas. Where windows, intersecting walls and other components are installed later, maintaining continuity of the building envelope, particularly the air barrier, is fairly readily accomplished. Therefore from a construction point of view there is a strong rationale for using this simple arrangement of wall components.

From a theoretical point of view, location of the insulation, air, and vapour barrier on the cavity side of the backup wall is the correct decision in terms of minimizing thermal bridges, and keeping the backup wall warm and therefore not subject to the effects of thermal deformation cycles. Also if the quality of the initial construction and the long term durability of the materials can be assured, superior thermal insulation, air barrier, and vapour barrier materials and/or details can be employed. This construction scheme also avoids in-use damage such as installation of wall mounted fixtures and cracking due to building movement.

## 10.6 Vulnerabilities

It is recommended that every design should be subjected to a vulnerability assessment including the following:

- Use of insulation on the inside of the CM wall increases the likelihood of cracking of the wall due to *unintentional restraint to thermal movements.*
- It is difficult to achieve an airtight seal on both face shells at joints. Therefore air which leaks into the cells of the blocks can circulate through the units and exit at these joints.
- Shrinkage of the concrete block can result in cracking due to restraint or opening up of sealed joints as a result of shortening.
- Unless parging is used, the sometimes uneven surface of blockwork may make it difficult to place insulation tight against the wall.
- For rigid insulation available in precast widths, the fixed positions of ties protruding from the mortar joints in the CM backup wall may cause difficulty with achieving a tight fit at joints between the insulation. Use of joint reinforcement as the tie system may further complicate proper placement and continuity of the sheets of insulation particularly where the joint reinforcement has been bent.
- When the BV and CM walls are built at the same time, mortar droppings from both sides of the cavity increase the volume of droppings and the possibility of impeding drainage of the cavity.

## 10.7 Key Construction Requirements

From the previous discussion of BV/CM building science requirements and other considerations, the following key construction requirements for construction of the system stand out:

- Since the CM backup forms part of the wall's moisture resistance, good workmanship regarding filling and tooling of mortar joints is required to achieve the intended airtightness and watertightness of the wall.
- Where parging is specified, it must provide a continuous surface at least 12 mm thick.
- At movement joint locations between the CM backup and the structure, sealing of these gaps requires special care to achieve continuity of air and vapour barriers.
- Regardless of the location of the primary air barrier, joints and block penetrations must be sealed to prevent air circulation within the CM wall.
- Fastening of the insulation must ensure that the insulation is held tight to the backup wall and that continuity of insulation is maintained.

- To minimize shrinkage and assure improved bond, only dry blocks should be used in construction.
- Both during and after completion of masonry construction, the masonry must be protected from weather effects as specified in CSA Standard A371 "Masonry Construction for Buildings".

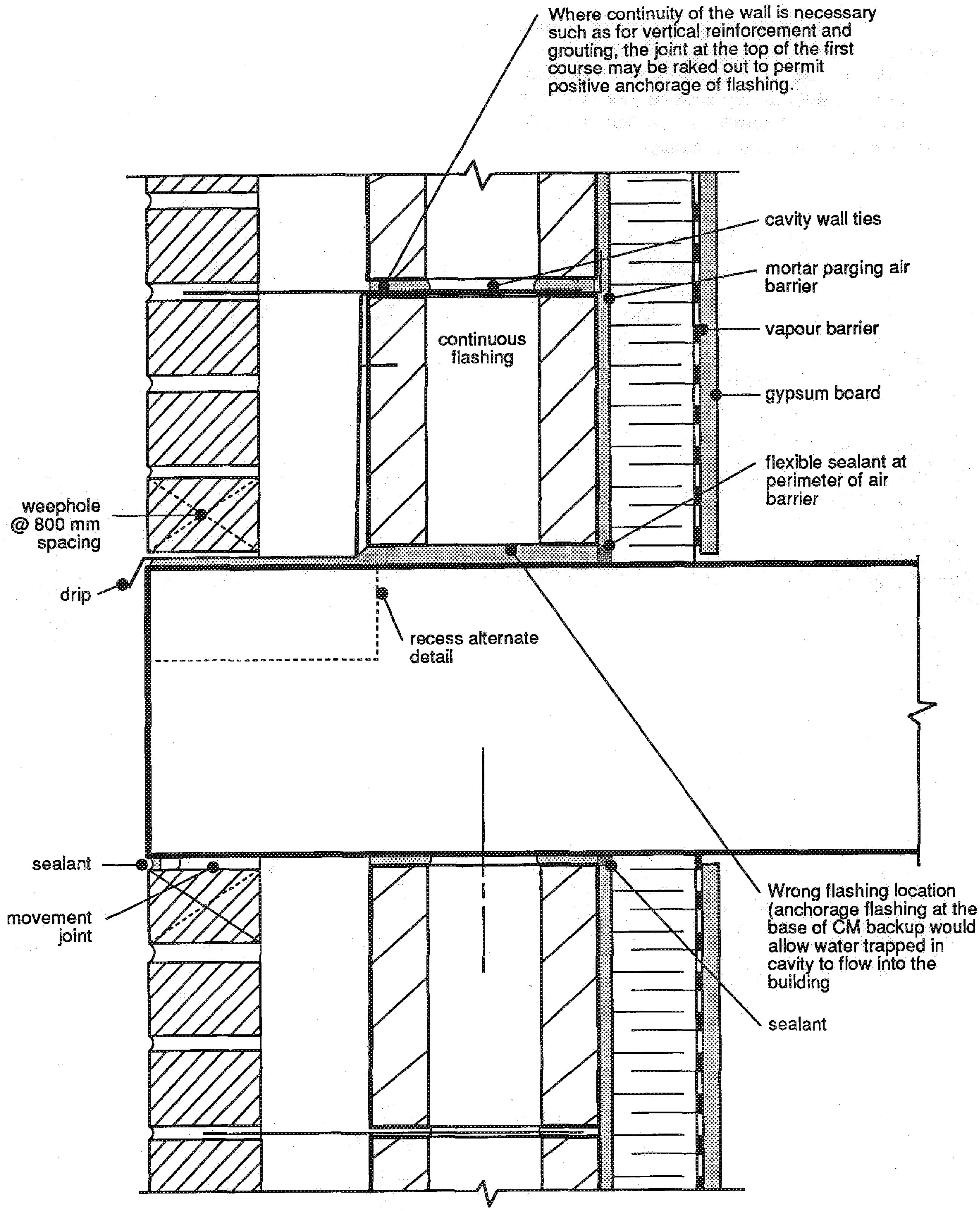



Fig. 10.1 Flashing detail for CM backup wall.

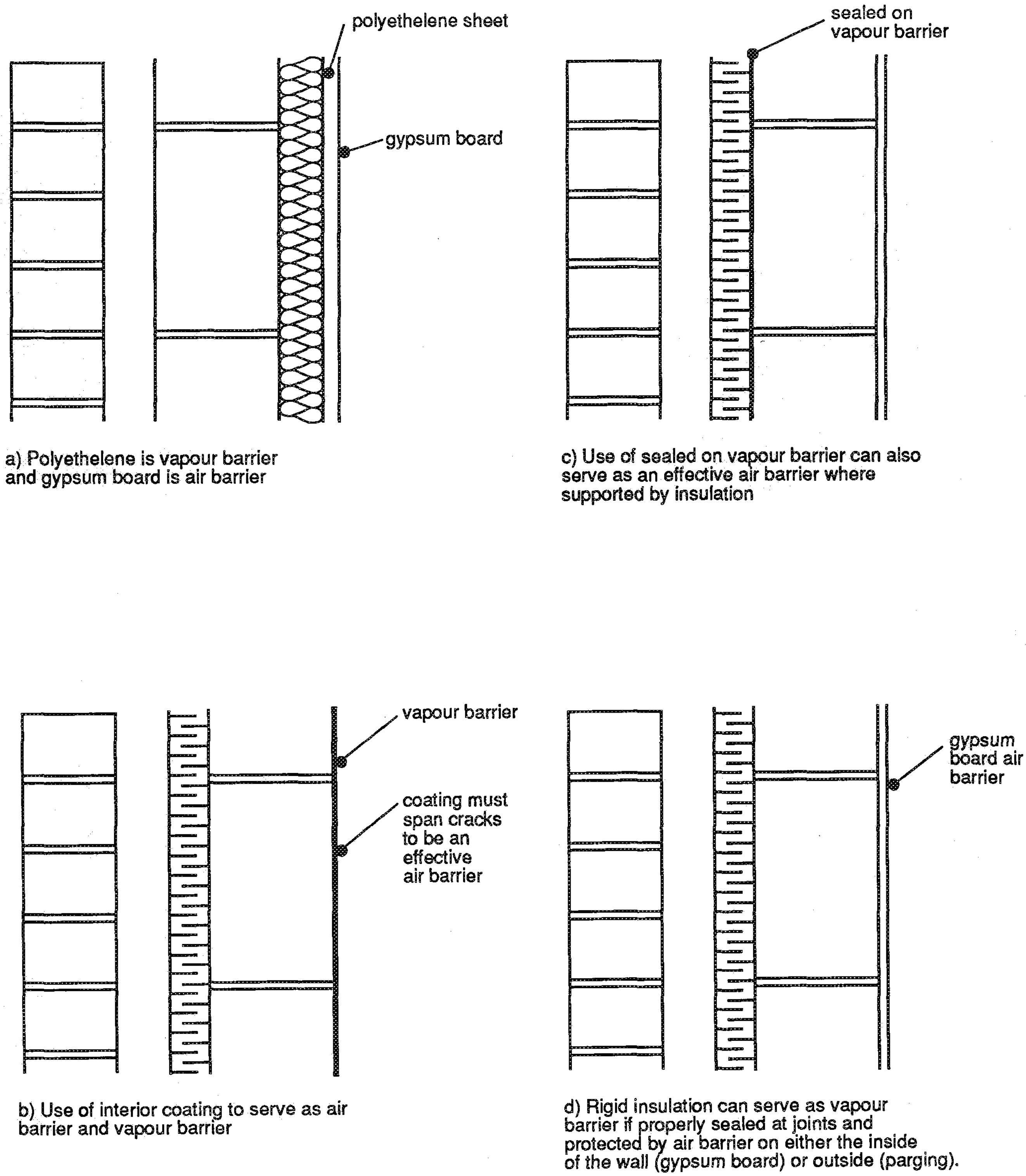



Fig. 10.2 Various locations of vapour barriers and air barriers for the CM backup wall system.

## CHAPTER 11
## BV/SS SYSTEM: BUILDING SCIENCE REQUIREMENTS

### 11.1 Provision of a Weather Screen

As discussed in Chapter 9, properly built brick veneer can be resistant to rain penetration with only small leakage paths existing. However, where the veneer is tied to the relatively flexible SS backup wall, wind pressure can cause very high flexural stresses in the veneer. Whether or not the brick veneer cracks will depend much more on the flexural tensile bond strength between the mortar and the bricks than on the strength of the SS backup wall. To preclude flexural cracking at full design load, design of the backup wall to be sufficiently stiff to take most of the wind load would render this form of construction uneconomical for most applications.

In designing the SS system to carry all of the wind load, such as basically happens after cracking of the veneer, the deflection limits should satisfy acceptable masonry crack width criteria. A simplistic and slightly conservative method for prediction of crack width is illustrated below:

From Fig.10.1, if the lengths, $\frac{L}{2}$, of brick veneer on either side of a crack are assumed to remain straight, the geometric relationship between mid-height deflection on the veneer, $\Delta_V$, and the maximum width of the crack, $\Delta_C$, is

$\Delta_C = 4\Delta_V \frac{t}{L}$ where t is the thickness of the veneer.

Then if the allowable deflection for $\Delta_V$ is $\frac{L}{C}$ $\quad \Delta_C = \frac{4\,t}{C}$ where design practice has tended to adopt C values between 360 and 720. For example, if a deflection limit of $\frac{L}{600}$ is specified for use of 90 mm thick brick veneer, the maximum width of the crack would be

$$\Delta_C = \frac{4(90)}{600} = 0.60 \text{ mm.}$$

At present there are no serviceability criteria regarding allowable crack widths for brick veneer. However, it seems obvious and has been demonstrated that cracking does result in substantial increases in penetration of wind driven rain water. In addition to water penetration, there is concern that sufficiently large cracks could result in freeze-thaw damage. For lack of other information, the crack widths allowed in normal reinforced concrete can serve as a guide. Under service conditions, reinforced concrete crack widths between 0.1 and 0.2 mm are a maximum. Therefore, even if the average crack width is used as the basis, it would seem that an allowable deflection of $\frac{L}{360}$ resulting in a maximum crack width of 1 mm for a 90 mm brick or an average of 0.5 mm is a rather large value. For $\frac{L}{720}$ the average crack width of 0.25 mm is closer to the concrete comparison but is still relatively large.

In view of both the previous discussion and the absence of crack width criteria for BV, the designer must select a design SS stiffness with due regard to the environmental conditions to be encountered by the structure. <u>In general, it is recommended that the SS stiffness not be less than L/720</u>.

Unless construction procedures are employed which assure that water penetrating the veneer cannot cross the cavity (ie. mortar fins and mortar droppings are removed), it is necessary that the exterior sheathing have some water resistant properties so that water will run down rather than be absorbed by the sheathing. In addition, when ties or other components puncture the sheathing, these areas should also be effectively sealed to exclude water. Also sealing of these paths is particularly important since ties may serve as cold thermal bridges and become potential locations of condensation of exfiltrating airborne moisture. Because of this potential availability of water, these areas require special attention to avoid corrosion and damage to exterior sheathing.

The flashing must satisfy the requirements discussed in Section 9.4. In this regard when exterior building paper is not used over the exterior sheathing, the flashing should extend up behind the sheathing as shown in Fig. 11.2. This can be an effective method for holding the flashing in place. When flashing is placed under the building paper or otherwise attached to the outside of the exterior sheathing, constructible details must be developed which assure that water cannot run down behind the flashing.

### 11.2 Resistance to Heat Flow

One of the characteristics of SS backup walls is that the inter-stud cavity can be used to house insulation resulting in saving of floor area. In almost all cases, batt insulation is used between the studs where a friction fit keeps the batts in place. For batt insulation to be effective, convection must be avoided. Therefore the inter-stud space must be completely filled. Where double studs are used or where corners are formed, special attention is required to assure that the space enclosed between studs is filled with insulation.

Tests, analyses and observations confirm that the thermal bridge formed by the steel stud does result in lower winter wall temperatures directly over the stud. This can result in surface marking as well as heat loss. Additional insulation, usually in the form of rigid insulation on the exterior of steel studs, can be added to the wall for the following reasons:

- decrease overall heat loss by increasing the thermal resistance
- increase the temperature of the SS assembly by making it a "warm" thermal bridge. This eliminates interior surface marking and, because the steel components are warmer than the surrounding materials, condensation in the wall is less likely to accumulate on the steel components.

### 11.3 Resistance to Vapour Transmission

The vapour barrier should be on the warm side of the insulation or at least inboard of the dewpoint. For most applications, this means that the vapour barrier will be located at the inside face of the studs. The two most common forms of vapour barriers are:

- 0.15 mm thick polyethylene sheet lapped over 2 lines of studs at joints or otherwise effectively sealed at joints. Because the polyethylene sheet has some resistance to air flow, it must be supported over its entire surface area.
- The interior sheathing may itself form the vapour barrier. Taped joints in the gypsum board and use of two coatings of quality latex paint or use of vinyl wallpaper will be adequate for most residential interior environment conditions.

In cases where rigid insulation is used on the outside of the backup wall, this insulation may itself provide an effective vapour barrier. However with 90 mm thickness of batt insulation in the cavity, the dewpoint will normally lie within the batt insulation unless an uncommonly large thickness of rigid insulation is incorporated into the design. In lieu of more accurate calculations, a rough rule of thumb is that 2/3 of the thermal resistance should lie outside of the vapour barrier. Otherwise condensation of moisture will occur at the inside of the rigid insulation potentially leading to:

- wetting of the batt insulation,
- accumulation of frost and water on the inside of the rigid insulation or exterior gypsum board.
- in extreme conditions, mildew and mold can form which has health implications as well as durability effects.

### 11.4 Resistance to Air Flow

Sound positive and negative arguments can be made for locating the air barrier either on the inside or the outside of the SS wall assembly. Again it is suggested that an assessment of the vulnerabilities for particular situations may help in making the decision. (See Fig. 11.3)

If the gypsum board on the interior of the SS assembly is used as the principal air barrier, it has the following positive aspects:

- The air barrier and all its components are maintained at warm room temperatures and therefore may be less susceptible to opening of cracks due to thermal movement or embrittlement of sealants.
- The air barrier is relatively easy to inspect and faults can be corrected at almost any stage of construction since the barrier is not covered over with subsequent layers of construction.
- Similar to the points above, maintenance or repair may be readily carried out on the interior wall covering. This includes replacement of caulking where it is used as part of the air barrier.
- Continuity of the air barrier with window frames and other through-the-wall openings may be more easily accomplished where flashings above and below the window make it difficult to achieve a permanent seal at the outside of the SS backup wall.

The following are the main negative factors:

- Occupant use such as hanging pictures and drapery track may puncture the air barrier.
- Installation of services such as electrical outlets may provide large, difficult to seal punctures through the air barrier.
- Movement joints, particularly at the track which allows for vertical movement, may be difficult to seal because aesthetics may limit the use of effective materials or construction techniques.

If the exterior sheathing is used as the air barrier, it has the following advantages:

- When completed, the air barrier is unlikely to be damaged by subsequent building trades or by occupants
- Use can be made of newly developed materials such as stick-on, torch-on, or spray-on sheets as strips to seal joints between the exterior sheathing and between the exterior sheathing and the surrounding structure.

The following disadvantages apply:

- The construction is not conveniently inspected and unless full time inspection is employed or construction is delayed until inspections are completed, significant parts of the air barrier are likely to be covered up prior to adequate inspection.
- Long term maintenance and repair of deteriorated areas and/or faults is virtually impossible. Therefore high quality construction must be guaranteed and materials must have excellent long term performance characteristics.
- Ties, screws, and other wall fixtures may be very difficult to seal.

To create an effective air barrier it is important to detail effective and constructible methods of handling continuity of the air barrier at:

- bottom track
- movement joints (principally at the top of the backup wall for vertical movement)
- windows and other openings
- connections to partitions and other parts of the structure
- locations where the barrier is punctured by services or other attachments.

## 11.5 Location of Wall Components

From a theoretical point of view it is well established that the vapour barrier must be located on the warm side of the insulation or, where several layers of insulation are used, it must be positioned in board of the depth representing the dewpoint location. Similarly it is fairly well accepted that to the extent possible, insulation on the outside of the backup wall is most effective since it helps keep the components of the backup wall warm. Minimizing thermal deformations, prevention of condensation, and moderate temperatures should all aid in maintaining the integrity of the air and vapour barriers. Because the steel components of the backup wall are zones of thermal bridging, it is important that a strategy be adopted to minimize this effect.

While placement of insulation on the inside of the steel backup wall is feasible, it will result in the steel stud being much colder and therefore more prone to condensation of airborne moisture. Figure 11.4 illustrates the temperature distribution at the interior face of exterior sheathing for several different configurations of thermal insulation. The dewpoint temperatures for air exfiltrating from humidified buildings are also shown for interior air at 20°C and 30,40 and 50% relative humidities. As can be seen, use of only exterior insulation results in the wall being least susceptible to condensation whereas the more traditional stud cavity insulated walls or a wall with an interior layer of insulation are most vulnerable to condensation of moisture from exfiltrating warm air. In these latter cases, having a very effective air barrier is essential to avoid the detrimental affects of condensation.

If it is assumed that some exfiltration of moisture laden air will occur, then it seems reasonable to choose insulation and insulation locations which will minimize condensation, particularly on critical components. In addition, for cases where condensation in the wall does occur, the potential for the wall to dryout should also be assessed. The decision on the location of the air barrier should be influenced by practical considerations such as inspection requirements, repairability, long term durability and maintenance. Therefore, location of the air barrier at either the outside or inside surfaces of the SS backup wall may be appropriate for particular situations.

## 11.6 Vulnerabilities

It was recommended that every design should be subjected to a "vulnerability assessment" including the following:

- likelihood of cracking of the veneer and the influence of potentially greater leakage on performance of the wall
- construction conditions and the effectiveness of quality control to produce a near perfect air barrier. The choice of BV/SS components must be tied to the level and frequency of inspections required to achieve satisfactory performance
- long-term durability and ability to carry out regular maintenance
- support for vapour barrier if it is a separate component from interior sheathing
- air barrier must be designed to be continuous. Movement joints must be effectively spanned by extensions to both air and vapour barrier
- the air barrier/vapour barrier design should be evaluated as a potential double barrier system where air leakage may occur
- design and construction provisions must ensure that moisture due to exfiltration or infiltration does not expose metal components to repeated wetting and potential corrosion.

## 11.7 Key Construction Requirements

From previous discussions of BV/SS building science requirements and other considerations, the following key construction requirements for construction of the system stand out:

- The quality of the veneer construction including filtering and tooling of joints is of great importance to achieve a sound first barrier against rain penetration.
- Flashing must be securely and continuously attached to the backup in order to intercept and direct water out of the cavity.
- In addition to proper installation of air and vapour barriers over the wall surface, continuation of these barriers at upper and lower tracks, windows and ends of walls requires special attention to prevent damage due to build-up of condensation and ice.
- Batt insulation in the intra-stud cavity must fit snugly to eliminate air circulation or sagging of the insulation.
- Spaces between double studs or other stud wall assemblies must be filled with insulation during construction to minimize the effects of thermal bridging.
- Work by various trades must be co-ordinated to prevent damage to completed work.

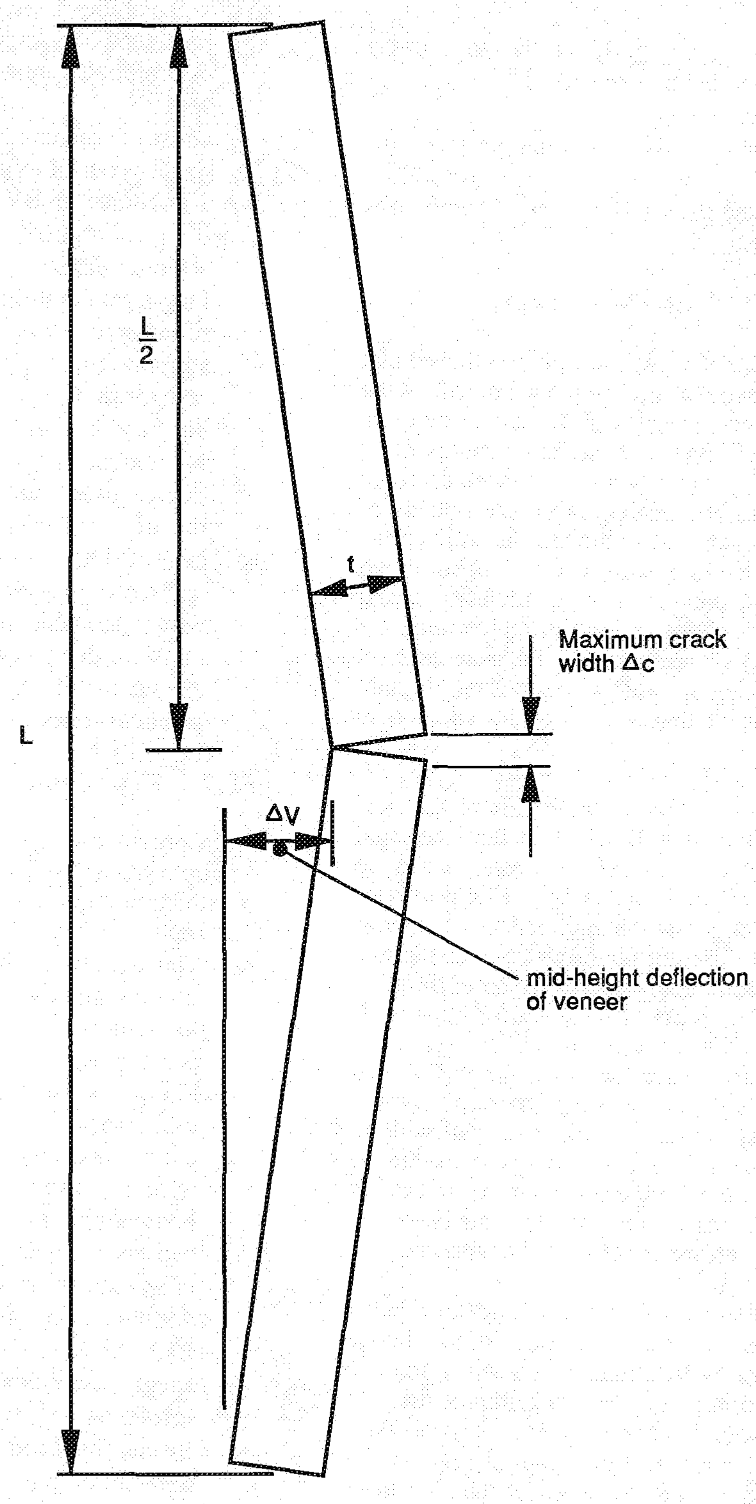



Fig 11.1 Schematic drawing for simplistic calculation of crack width in brick veneer.

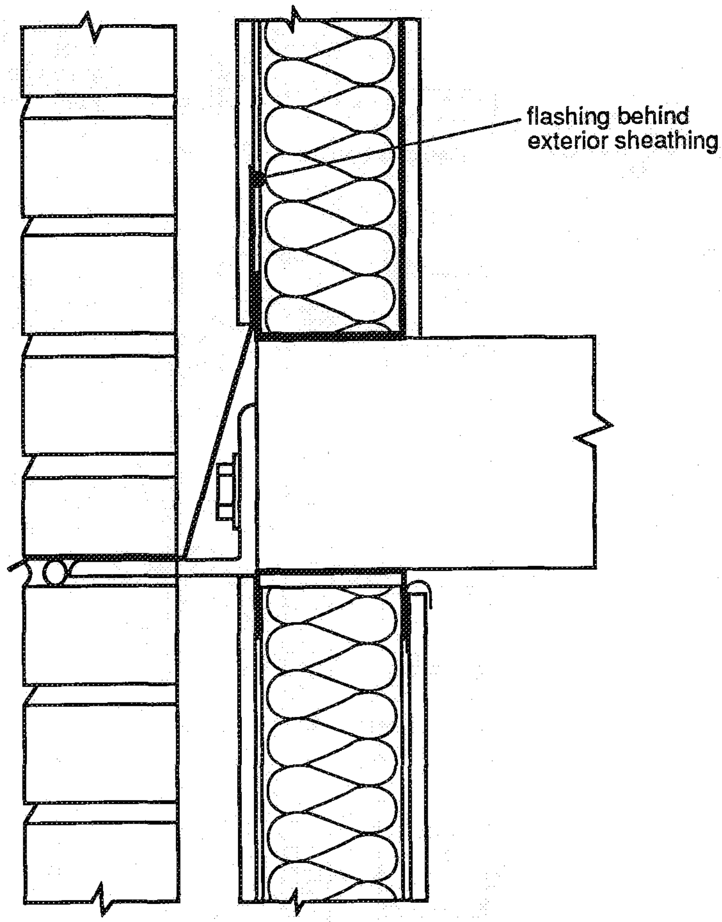



Fig. 11.2 Location of flashing behind exterior sheathing.

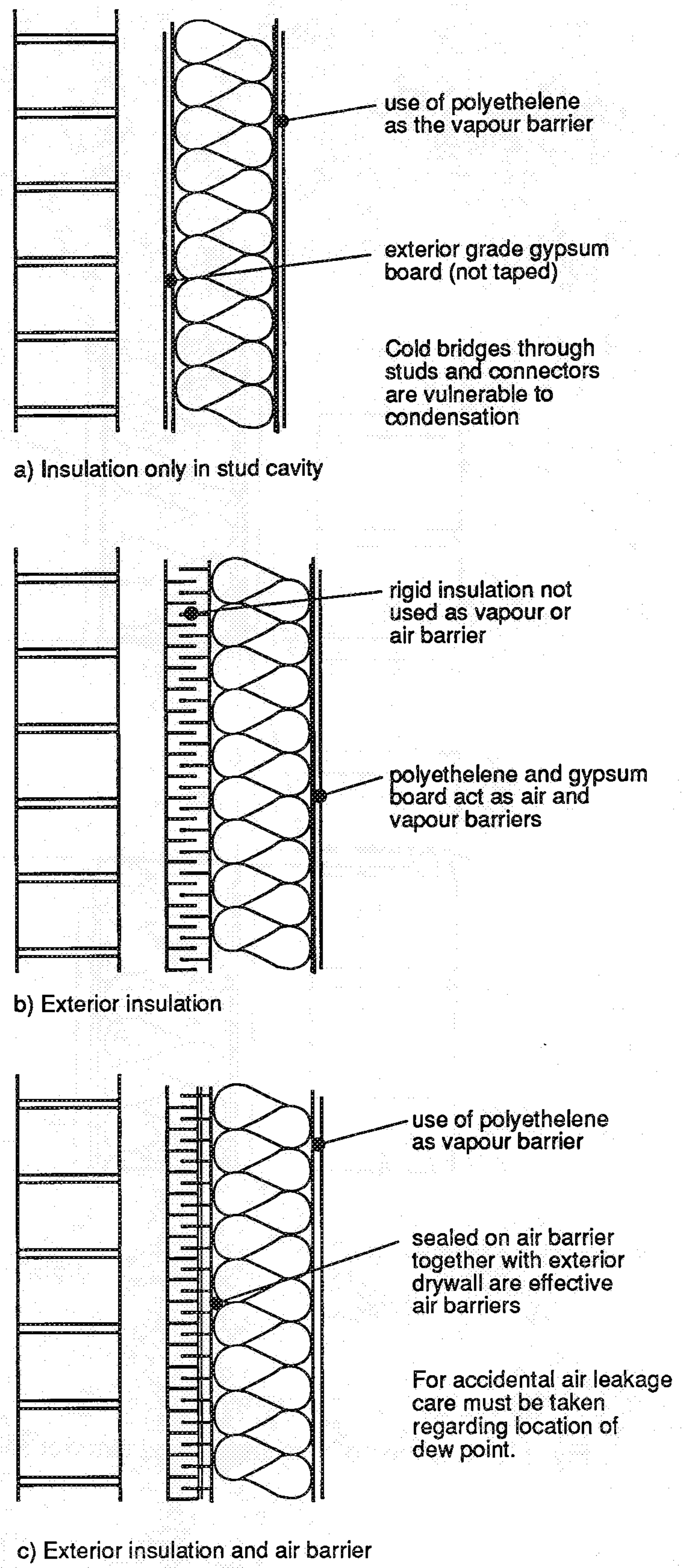
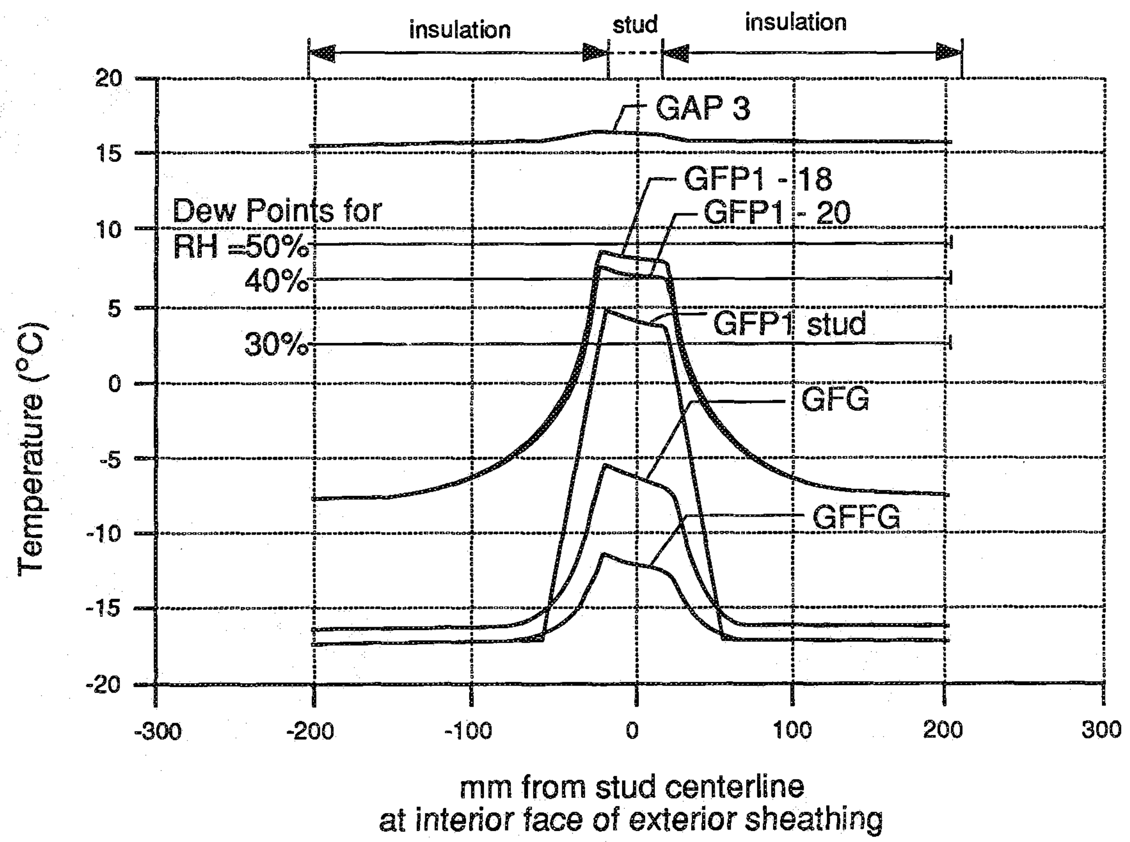



Fig. 11.3 Various locations of vapour barriers and air barriers for the SS wall system.




GAP3 = wall with no insulation in stud space and 75 mm rigid insulation on the outside

GFPI-18 and GFPI-20 = wall with fiberglass in stud cavity and 25mm polystyrene exterior sheathing. 18 and 20 guage studs were used.

GFPI stud refers to the case where rigid insulation was just put over the exterior flange of the stud

GFG is the standard wall with no exterior insulation

GFFG is the standard wall but with a 25 mm layer of fibreglass on the interior face of the stud/

Fig. 11.4 Temperature profiles for various wall configurations.

# CHAPTER 12
# MOVEMENT JOINTS

## 12.1 General

A variety of materials are used in the construction of veneer wall systems and the main structure. These materials such as brick masonry, concrete block masonry, concrete, and steel exhibit different deformation characteristics under load and under changes in thermal and moisture conditions. Moreover, the various components of the veneer wall system and main structure are subjected to different loading, thermal and moisture conditions. Resultant differential movements can lead to cracking, spalling and collapse of veneer panels unless movement joints are provided at strategic locations to control potential buildup of stresses.

## 12.2 Material Deformations and Building Movements

The following key material deformations are to be considered:

- Elastic deformation
- Long-term deformation
- Moisture movements
- Thermal movements

The changes in length may seem quite small but over long lengths can be sizeable. For example, an unrestrained concrete block wall 30 m long would shorten about 10 mm because of shrinkage. Other movements are of the same order of magnitude. Movements may be additive or take place in opposite directions, partially cancelling out. For example, moisture and temperature movements can be in the same or opposite directions. In general, it is movements which are restrained or differential movements between connected elements that cause problems.

## 12.3 Width of Movement Joints in Clay Brick Veneer

The required width of vertical movement joint is primarily governed by the maximum expansion or contraction which can occur in a given segment of veneer as well as by the properties of the sealant and backing material to be used to make the joint weathertight. The maximum expansion or contraction with respect to the position of the veneer during application of sealant is governed by the veneer temperature during sealant application and the time interval between construction and sealant application. By the time sealants are applied, some of the brick moisture expansion has taken place already but this will not be discounted in the following example. When the sealant installation temperature is 20°C, the maximum expansion in a 10 m long veneer is 4.7 mm whereas prior to moisture expansion, the contraction is 2.7 mm. On the other hand, if sealant is applied when the veneer temperature is 5°C, the maximum expansion is 5.6 mm and the maximum contraction is 1.8 mm prior to moisture expansion of the brick. The width of the joint is governed by the arithmetically greater value of expansion or contraction. Therefore, for minimum joint thickness, the sealant ideally should be applied when the veneer temperature is about halfway between the extremes.

Widely used sealants for masonry applications are silicones and polyurethanes having a cyclic deformability of about ±25%. If such a sealant is to be applied when the veneer temperature is 20°C, the required width of movement joints spaced at 10 m is about 19 mm. However, if the veneer temperature is 5°C during application of sealant, a 22 mm joint would be required.

## 12.4 Horizontal Joints for Vertical Movements

As illustrated in Fig. 12.1, the concrete frame of a 10-storey building may typically shorten by 15 mm due to elastic deformation, creep and shrinkage. On the other hand, the clay brick masonry veneer may typically expand by about 15 mm due to moisture and thermal expansion. The resulting differential movement of 30 mm would severely strain

- veneer ties and window frames in the upper floors of the building

- roofing membrane, flashing and parapet details.

To prevent such distress, the veneer typically is supported at each floor level and a horizontal movement joint is provided as shown in Fig. 12.2 for a steel shelf angle support system. Assuming a veneer storey height of 3 m, the vertical differential movement between veneer and frame is about 3 mm, an amount which can be readily accommodated in a reasonably sized movement joint. Using a sealant with a cyclic deformability of ±25% and keeping in mind construction tolerances and slab deflections, a 10 to 15 mm movement joint typically is adequate. Note that in Fig. 12.2 the movement joint requirement refers to the clear vertical gap between the underside of the shelf angle and the top surface of the veneer. Depending on the thickness of the leg of the shelf angle, the apparent joint width would be greater and typically of the order of 20 to 25 mm.

A movement joint is also required between non-loadbearing backups and the floor slab or spandrel beam above to accommodate deflection of the slab or beam, frame shortening and thermal expansion of backup; this movement joint will prevent any transfer of vertical load to the backup. SS backups designed to resist only lateral loads are particularly vulnerable to distress if such load transfer occurs.

## 12.5 Vertical Joints for Horizontal Movements

Movement of masonry veneers also occurs in the horizontal direction. Assuming such movement to be unrestrained, a situation illustrated in Fig. 12.3(a), a 15 m long clay brick wall laid up at 20° C could undergo a free expansion of 7 mm. This amount of expansion is based on a moisture expansion of 0.02% and a surface temperature increase of 45° in a hot summer sun. Note that since a wall expands from its centre as indicated in Fig. 12.3(a), it will move about 3.5 mm both ways.

If, instead of being free, the brick wall was on a concrete foundation, the brick wall will then take a shape somewhat as shown in Fig. 12.3(b). Generally there will be some slip towards the end of the wall and the brickwork will overhang the foundation a small amount. In slipping, it usually tears away a small piece of concrete as illustrated in Fig. 12.3(c). In even longer walls with openings, the strains could cause the type of cracking shown in Fig. 12.3(d).

If the temperature of the brickwork subsequently falls to —25°C or lower in the winter, the overall reduction in temperature will be about 90°C. This large drop from extreme summer conditions will shorten the brickwork 4 mm at each end, and the wall will tend to take the shape shown in Fig. 12.3(e), unless the base of the wall is able to slip along the foundation. Generally, however, there will be some roughness between the concrete and the brickwork, and the restraint of movement can cause large tension forces. Brickwork is weak in tension and one or more cracks may occur in the wall.

To minimize cracking, in long walls it is advisable to provide expansion/ contraction joints at frequent intervals.

In the case of highrise structures, the base restraint at each support level effectively means that the veneer is made up of a series of lowrise walls and therefore can be treated in a similar fashion to lowrise walls.

Further discussion of vertical joints for horizontal movements may be found in various references. The Brick Institute of America, Technical Note on Brick Construction, No. 18, recommends that they be placed in long walls and at offsets, junctions, corners and parapets as illustrated in Fig. 12.4.

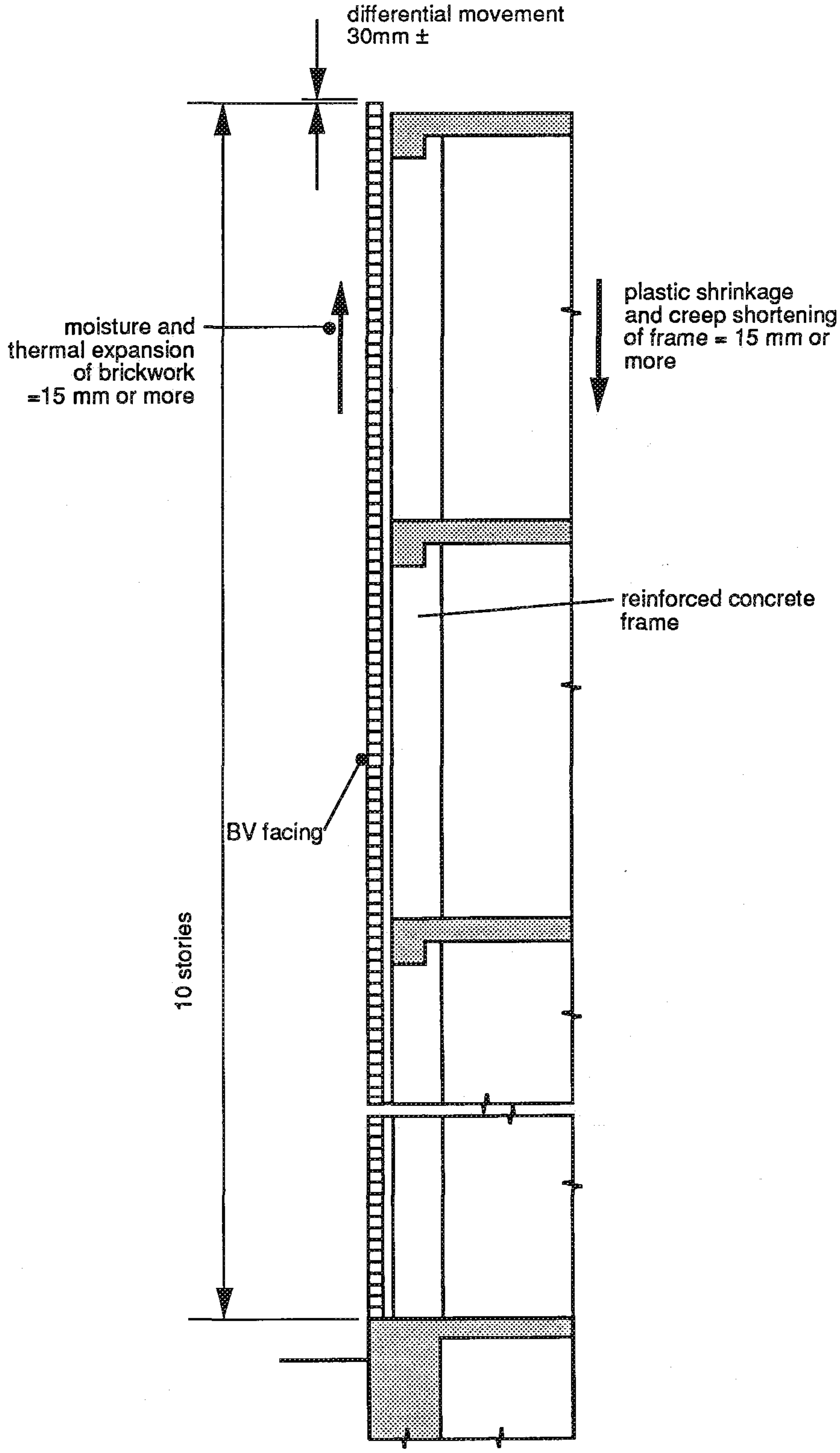



Fig. 12.1 Difference in height of clay brick veneer and frame if relative movements unrestrained.

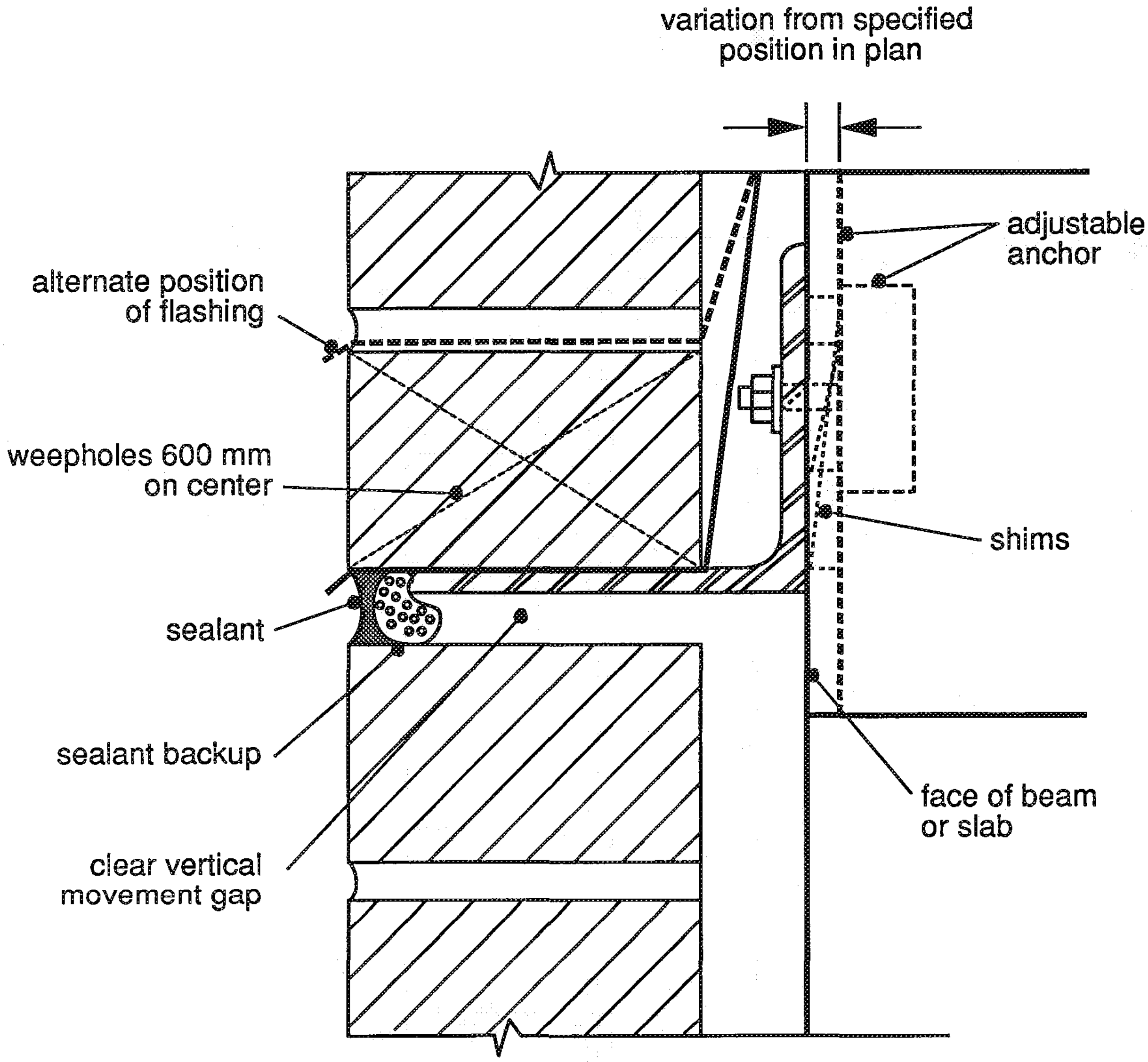



Fig. 12.2 Detail of horizontal movement joint at shelf angle. While some designers specify "soft" filler strips in the movement gap, such practice is not recommended for two reasons: some filler strips can transfer significant compressive loads from structure to veneer and an open gap can be more readily inspected.

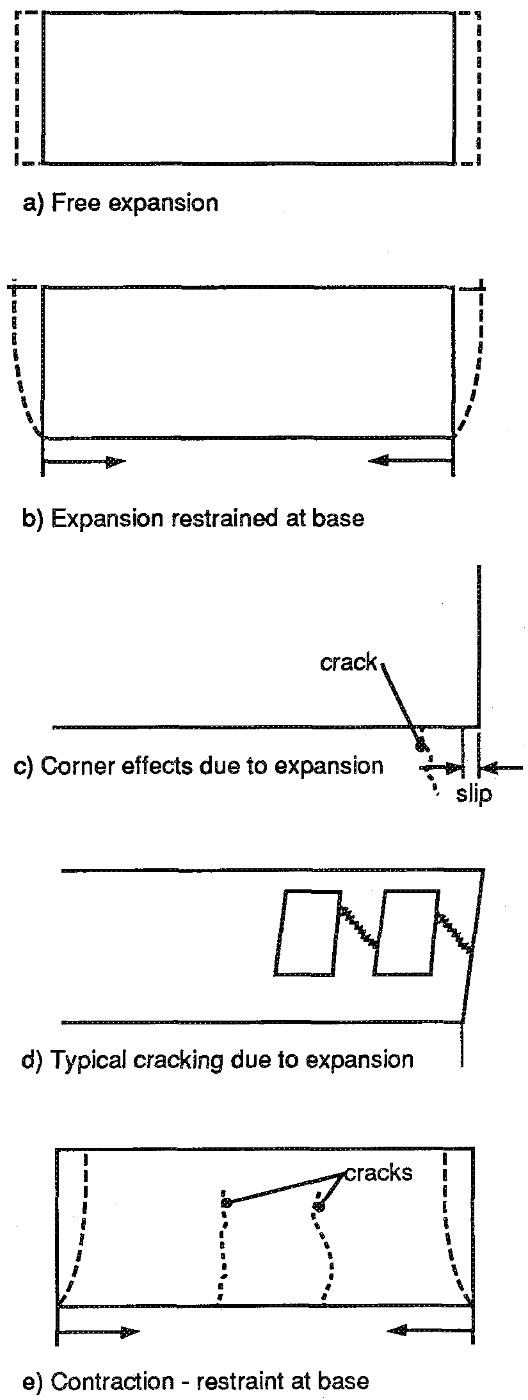



Fig. 12.3 Effects of expansion and contraction on long masonry walls

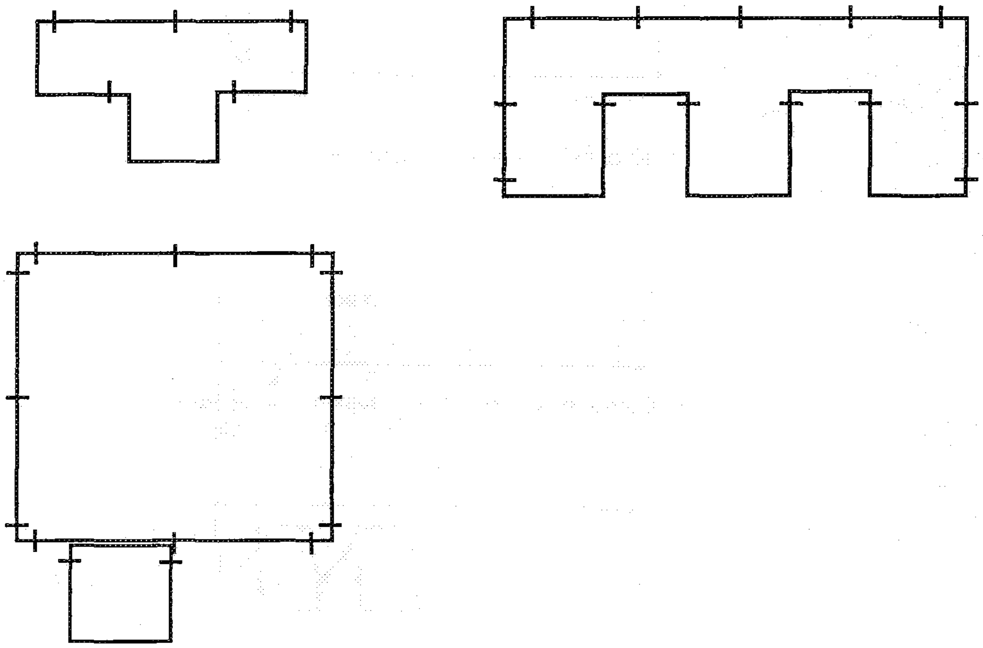


Fig. 12.4 Typical locations of movement control joints.

# CHAPTER 13
# PARAPET AND ROOF CONNECTIONS

## 13.1 Introduction

Parapets are one of the most exposed parts of a building, being subject to wind driven rain and temperature extremes on both wall faces. Water penetrating parapets and draining into walls below has been responsible for moisture problems. Their proper flashing has been discussed professionally for many years. A variety of materials are used and there is no standard parapet design. Some designers prefer to omit parapets whenever possible and simply use well-flashed curbs at roof edges.

## 13.2 Details

CSA Standard CAN3-A371 M84 stipulates that parapets be constructed of solid or grout filled hollow units. Parapets are also required to have through-the-wall flashings and, if they are less than 300 mm thick, to be flashed on the back for their full height, or at least 900 mm above the roof level. This is to take care of the likelihood of snow drifting against the parapet. Fig. 13.1 shows such a parapet.

Instead of a parapet, many highrise residential buildings today employ a metal cap as illustrated in Fig. 13.2. Simple to construct, such a detail can achieve a satisfactory continuity of the building envelope between veneer and roof.

Key Construction concerns

For veneer, provide mortar free movement joint

Install flashings and counter flashings with great care since parapet is typically the most exposed building element.

Achieve complete continuity between roof membrane and flashing.

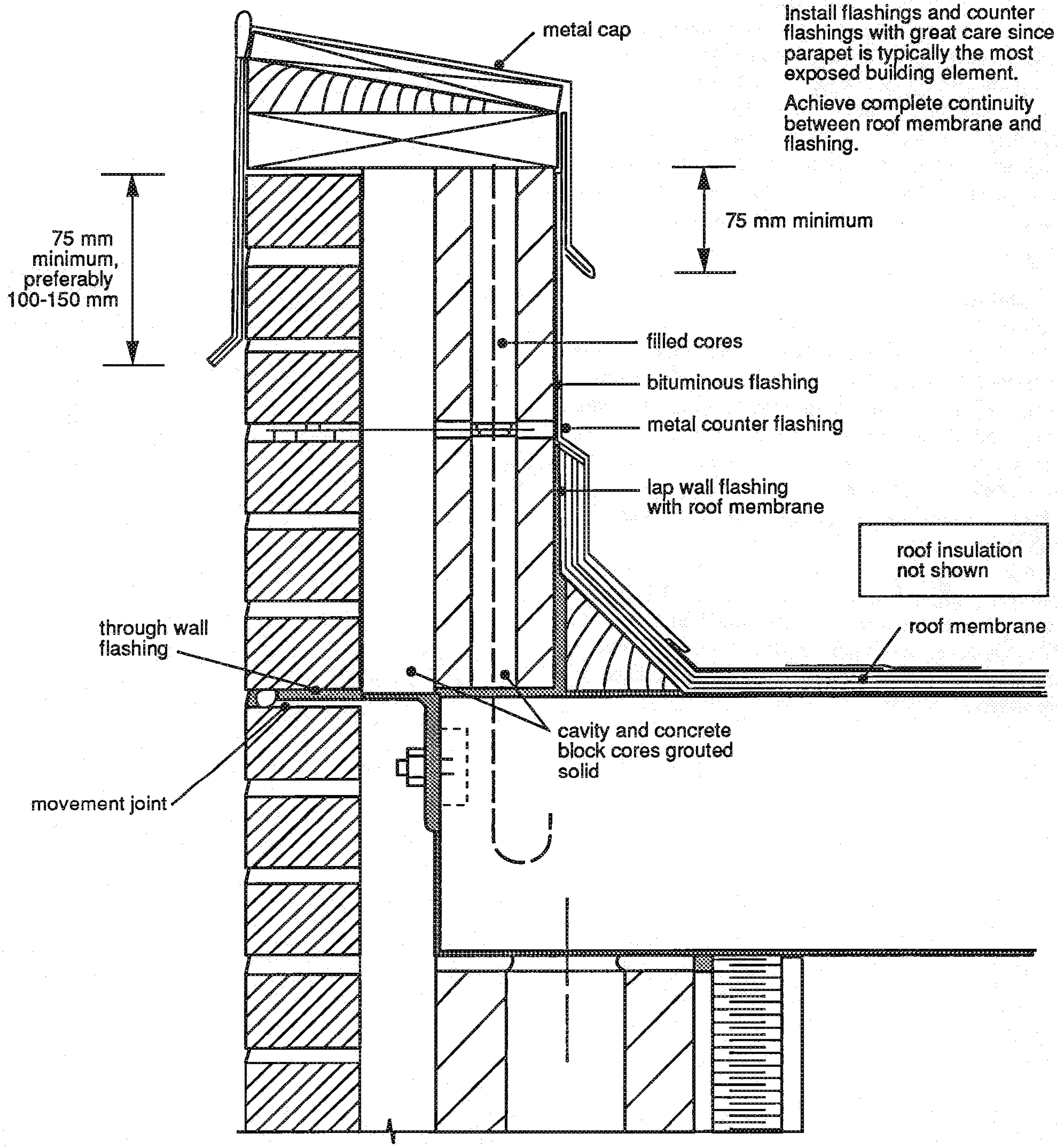
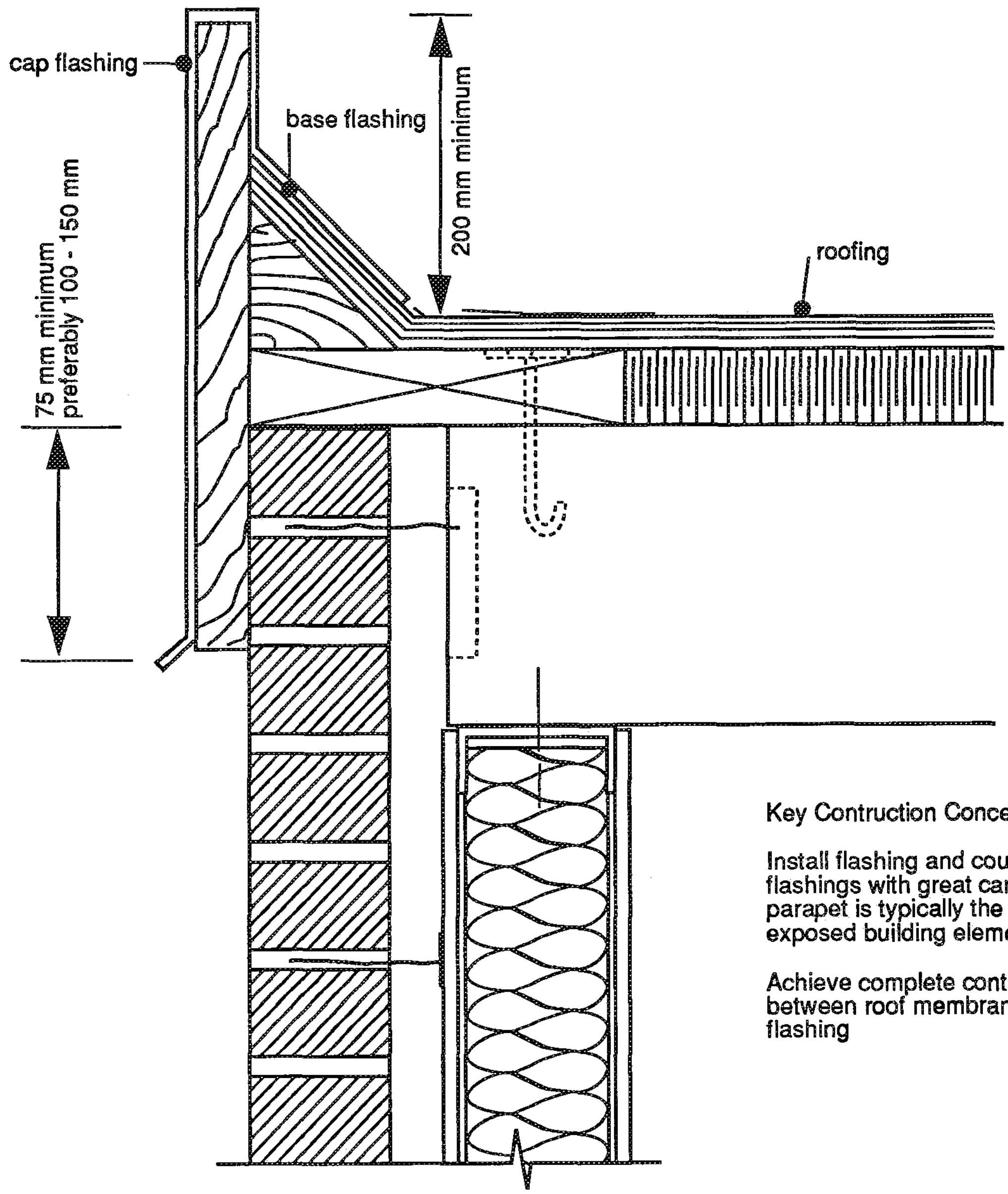



Fig. 13.1 Parapet flashing




Key Contruction Concerns

Install flashing and counter-flashings with great care since parapet is typically the most exposed building element.

Achieve complete continuity between roof membrane and flashing

Fig. 13.2 Flashed curb

# CHAPTER 14
# EFFLORESCENCE

## 14.1 Introduction

Many kinds of water soluble salts are present in mortar and masonry units. Where contrary to good practice, masonry is in direct contact with soil or a dampproofing course is omitted, such salts can also be carried into the masonry by moisture originating from the soil. As the moisture carrying water soluble salts evaporates, the salts crystallize and may form a usually whitish deposit on the exterior of the masonry; the effect is termed efflorescence.

Especially for cases of relatively severe efflorescence, the disfiguring staining may affect property and rental values and hence is annoying to owners. For efflorescence to occur, both water soluble salts and moisture must be present.

## 14.2 Sources of Salts

There can be many sources of salts including the veneer's brick units, concrete masonry backup units, mortar, building trim, ground water and atmospheric pollutants. Typically for a veneer, the main sources are the mortar and the brick units.

Regarding mortar as a source of efflorescence, suction between freshly laid units and the plastic mortar will transfer soluble salts present in the mortar to the brick. The salts may be present in the mortar from the cement, the sand, or from calcium chloride added to the mixing water.

Sodium and potassium hydroxides commonly present in portland cements may cause some measure of efflorescence in newly laid masonry during the first year. This efflorescence is known as 'new-building bloom' and the veneer is referred to as 'blooming'. Studies have shown that increasing the amount of portland cement in mortar increases the amount of efflorescence. Yet a given mortar showing pronounced efflorescence with one brick may not with another, and efflorescence is related as well, therefore, to the absorption rate of the masonry units. Presence of lime in mortar decreases the likelihood of efflorescence.

Both clay brick and concrete brick may contain soluble salts which can be brought to the masonry surface as migrating moisture evaporates. A standard test is available to determine the brick's capacity to contribute to efflorescence.

Building trim made of precast concrete, natural stone or cast stone may contribute significantly to efflorescence of adjoining veneer. Such materials may contain considerable amounts of soluble salts which can be leached out over long periods of time.

## 14.3 Sources of Moisture

As newly constructed masonry dries out, blooming may be displayed for a few weeks or perhaps several months. Typically rains will remove this efflorescence and in a season or two the blooming effect will have disappeared. However where efflorescence persists, a source of moisture should be looked for and controlled.

Rains, and particularly driving rains, can deposit substantial amounts of moisture in masonry walls. During periods of warmer weather, the rate of evaporation of moisture within a wall is relatively high and any soluble salts are therefore deposited within the brick veneer rather than on the surface. However during colder weather, the rate of evaporation is thought to be slow and moisture containing soluble salts can migrate to the veneer's surface prior to evaporation. In the Canadian climate, late fall, winter and early spring months hence represent the main season for recurring efflorescence problems.

Besides rain water, another major source of moisture can be leakage of moisture-laden air from a building's interior. Discontinuities in air and vapour barriers will permit such moisture-laden air to exfiltrate and condense on wall components during colder weather. Such moisture can then migrate by capillary action and deposit soluble salts on the veneer's surface.

## 14.4 Prevention

Since soluble salts in various concentrations are present in many materials used in masonry construction and also since moisture must be present to cause efflorescence, for new construction it is important that aspects of material selection, design, construction and maintenance are carried out with care so as to minimize the likelihood of efflorescence occurring on a building.

Concerning selection of materials, tests can establish that brick units do not effloresce and that mortars do not contain high solutions of water-soluble alkali. Use of mortar admixtures is generally not recommended because of their unknown ingredients and possible effect on masonry bond and on efflorescence.

Design using cavity wall construction and the open rainscreen principle is good practice as moisture penetrating the thin veneer can be vented and direct migration of salt solutions from CM backup is interrupted. Most important are design details which

- prevent the entrance of moisture into the wall system
- direct accumulated water away from wall tops and other wall surfaces.

Critical details include air and vapour barriers, copings, caps, sills, flashing, drips, vents and weepholes. Also critical are joints that are to be caulked, such as movement joints, and joints that are to be sealed, such as at window and door openings.

Construction must employ

- good workmanship practices to achieve the measure of efflorescence control intended in the design
- protection of the tops of walls under construction to control ingress of moisture
- proper storage of materials to ensure that moisture from rain and snow is kept away and contamination from salts is prevented.

Regarding maintenance measures to prevent efflorescence, the building envelope should be inspected periodically to spot moisture paths due to the deterioration of materials. Of critical importance is the proper maintenance of caulked joints and sealants.

## 14.5 Control

'New-building bloom' efflorescence will often disappear as repeated rainfalls rinse the soluble salts off the masonry surface. Where blooming persists or removal is required before natural weathering could do its work, rinsing with water while scrubbing with a brush is commonly employed. To help dissolve the salts, dilute solutions of hydrochloric acid are sometimes used. Two important points should be kept in mind when planning this type of efflorescence removal:

- The work should be carried out during periods of warm weather so that the moisture absorbed by the masonry during the flushing operation can evaporate quickly and does not bring additional salts to the surface.
- Treatments involving even dilute solutions of acid must incorporate flushing with water both before and after application of the acid solution. This will minimize the retention of acid within the veneer where it could lead to corrosion of embedded steel.

Where efflorescence is caused by abnormal wetting of the wall as from poorly located drains and scuppers, missing drips and coping or deteriorated caulking, these faults must be rectified to control efflorescence. Particularly difficult and expensive to remedy is efflorescence caused by exfiltrating air escaping through weaknesses in the air and vapour barrier systems. A careful assessment of design details and as-built conditions especially around openings is usually required before remedial actions can be formulated.

A number of surface treatments are available commercially to help control efflorescence. In essence these treatments typically impart various degrees of water repellency to the extreme outer layer of the masonry. While formation of efflorescence on the outer surface is suppressed partly due to less moisture entering the wall and partly due to evaporation of soluble salts taking place just behind the treated surface layer, serious problems may arise nevertheless: As the salts crystallize and accumulate behind the treated surface layer, pressures can be built up which lead to spalling of the masonry. Such surface treatments, however, can be successful if they are carried out in conjunction with other measures which will control the amount of moisture entering the masonry veneer.

## CHAPTER 15
## FREEZE-THAW RESISTANCE

### 15.1 Introduction

Although one of the outstanding attributes of brick masonry is its durability, cases do occur where the freeze-thaw resistance of brick is inadequate to withstand the relatively harsh Canadian environmental conditions. Resultant brick spalling is disfiguring and expensive to repair.

### 15.2 Code Criteria

CSA Standard A82.1-M87 "Burned Clay Brick" outlines grade of brick requirements according to a Canadian weathering index which for any locality is the product of the average number of freezing cycle days and the average annual winter rainfall. In essence the Standard requires that a grade SW (severe weathering) brick be used for veneer construction across Canada. For SW brick, the Standard permits acceptance of brick if the average saturation coefficient is less than 0.78, the average compressive strength exceeds 20.7 MPa and the average maximum 5-hour boiling water absorption is less than 17 per cent. It is noteworthy that the maximum value of 0.78 for the saturation coefficient is a new and considerably more stringent requirement than the value of 0.88 given in previous editions of the Standard. While the change was made to affect a reduction of freeze-thaw brick failures in the field, the criteria of saturation coefficient, compressive strength and absorption can only serve as indicators of later in situ performance.

It should also be noted that the saturation coefficient and 5-hour boiling requirements may be waived provided the brick passes 50 cycles of freeze-thaw testing with no breakage and no weight loss exceeding 0.5%. While the freeze-thaw test is regarded as a more reliable predictor of in situ performance, it simply indicates if a brick wetted to a certain degree of saturation and frozen at a certain rate for an arbitrary number of cycles will fail or not fail. In situ conditions may impose a much larger number of freeze-thaw cycles under differing climatic conditions which may be more severe than a standard freeze-thaw test.

### 15.3 Recent Research

Besides the saturation coefficient, which is the ratio of the 24-hour cold water absorption to the 5-hour boiling absorption, recent research has indicated that specific surface area and pore structure have a significant influence on clay brick durability.

A Division of Building Research (DBR) study of 27 bricks with known freeze-thaw field performance has indicated that a good correlation exists between specific surface area and performance: the brick was found to be durable for specific surface areas up to about 1 $m^2/g$ and generally unsound for larger values. From their work it is clear that underfired bricks would exhibit relatively large specific surface areas and consequently would be most prone to durability failures in the harsh Canadian climate.

A study of pore size distributions for brick indicates that pores can be classified into coarse, intermediate and small pores: coarse pores (greater than 1 micron) enhance durability, for although they readily absorb water, they also dry out rapidly; intermediate pores (say, between 0.1 and 1 micron) are most prone to cause freeze-thaw damage because they dry out more slowly than coarse pores yet are frequently saturated under service conditions; small pores do not fill or dry easily and additionally only freeze at quite low temperatures, hence do not represent a major force in causing freeze-thaw breakdown of a brick. The major force is seen to be the intermediate pore and in particular the per cent volume of such pores in relation to the other two sizes. If an assessment of total porosity shows a preponderance of intermediate pores, the brick will likely not be durable.

### 15.4 Best Practice

To minimize the potential for brick durability problems

- the brick's performance record over at least 5 years under varying exposure conditions should be checked
- the uptake of moisture in the brick should be controlled by careful detailing and workmanship as well as by regular maintenance measures
- brick in especially critical applications should be tested not only to meet the standard freeze-thaw durability require-ments of CSA A82.1-M87 but also non-standard requirements of specific surface area and pore structure.

It should be noted that surface coatings applied to control moisture uptake in veneer may act to some degree as a vapour barrier trapping moisture in the brick, thereby creating conditions susceptible to freeze-thaw damage.

# CHAPTER 16
# SURFACE COATINGS

## 16.1 Introduction

The main reason for applying coatings to masonry surfaces is to help control moisture movement. Masonry coatings can be classified either as solvent-based or as water-based coatings. The former contain or are soluble in organic solvents while the latter dissolve in or are dispersed in water. The successful performance of coatings depends on correctly assessing the cause of wall distress and then choosing the appropriate coating material both for the distress at hand and the masonry material properties.

## 16.2 Masonry Material Properties

In applying coatings to masonry, masonry material properties such as porosity, surface roughness, alkalinity and the presence of soluble salts are of key importance.

- Porosity: While clay brick is porous to small water molecules, coatings generally contain much larger molecules with the resultant effect that some denser bricks are relatively non-porous to coatings.

- Surface roughness: Since the chemical constitutions of the masonry substrate and coatings differ markedly, little chemical bonding takes place and adhesion depends mainly on mechanical keying. Most clay bricks exhibit considerable surface roughness for good adhesion; glazed bricks are an exception.

- Alkalinity: The main coating problem caused by alkalinity in the masonry has been a chemical reaction called saponification between oil-based coatings and alkali. Unless non-saponifiable coatings are used, a coating film can become soft and tacky. Clay brick normally being neutral, alkalinity in the veneer would be furnished by the mortar.

- Soluble salts: As discussed in Chapter 14, soluble salts may cause efflorescence. Efflorescence in turn may cause coating failure by mechanically destroying the film, the force exerted during crystallization of the salts being strong enough to overcome the bond to the substrate. Where efflorescence is encountered, it is important to remedy the source of moisture and not rely on a coating to perform a miracle function for which it is not designed.

## 16.3 Types of Coatings

A large variety of both solvent-based and water-based coating materials are available. Too numerous to list here in detail, they include latex paints, "breathing type" heavily applied coatings and silicones.

## 16.4 Best Practice

Whenever coatings are considered for masonry surfaces, it should be clearly understood that coating failure will take place when large amounts of moisture are able to enter the masonry through defects such as cracks or unfilled joints, poor detailing or moisture-laden exfiltrating air. As this moisture builds up behind the coating, adhesive failure of the coating occurs. Even breathing type coatings including silicones are not designed to handle the passage of larger quantities of moisture without distress. Under freeze-thaw conditions in particular, the highly saturated masonry may fail by delamination of the surface including the coating.

Additional best practice recommendations are the following:

- Coating performance on similar masonry in the same geographic and environmental region as the intended project application should be checked.

- In applying coatings to masonry it is important to closely follow manufacturer's instructions, otherwise coating performance may be seriously impaired.

- Since the service life of coatings varies widely depending on such factors as type of coating, type of substrate, geographic location and exposure conditions, a periodic checkup of performance is good practice. Re-application of coatings is typically required at 5 to 10-year intervals.

# CHAPTER 17
# MAINTENANCE AND REPAIR

## 17.1 Post-Construction Inspections

All building materials deteriorate to some degree with time due to weathering, aging and other environmental effects. Concerning durability of the BV/CM and BV/SS cladding systems, poorly maintained joints may for instance permit infiltration of excessive amounts of moisture. Since time of wetness is a key factor controlling rates of corrosion of steel, the frequent presence of excessive amounts of moisture may lead to premature corrosion distress in steel components of the cladding systems. Other cladding distress may be related to material problems such as unsatisfactory freeze-thaw-resistance of the brick, poor design detailing such as absence of movement joints, or poor construction practices such as inadequate attachment of a shelf angle.

Since most cladding problems only become apparent quite some time after completion of construction and also since initially minor problems may enlarge into major distress potentially endangering the safety of the public, owners must recognize the need for periodic post-construction inspections of building enclosures.

From an economic point of view it also makes sense to carry out periodic maintenance and minor repairs rather than allowing deterioration to proceed until major repairs or replacement of the cladding system are required. Figs. 17.1(a) and (b) illustrate this point schematically.

As shown in Fig. 17.1(a), the soundness of a cladding system reduces with time, first very slowly but during later time periods at an ever increasing rate. Assuming point B on the curve represents a degree of soundness which divides serviceability problems from safety problems, i.e. at soundness levels above B a safety problem is not present, then it is clear that periodic maintenance/minor repair actions are required at a time prior to $t_B$. If such actions were taken at a degree of soundness shown as point A in Fig. 17.1(a), the cladding system's serviceability problems would be remedied such that at a time $t_A$ the system again achieves 100% soundness similar to its original newly constructed condition. The deterioration cycle would then repeat itself until at some future date $t_C$ further maintenance/minor repairs would be initiated.

Fig. 17.1(b) indicates that maintenance/repair costs escalate significantly as the level of soundness of the cladding system decreases. In particular, the schematic curves show that very major costs are encountered once safety problems come into play and also that it is economically advantageous to commit maintenance/minor repair funds periodically rather than face much more major costs once the system is badly deteriorated.

## 17.2 Frequency and Nature of Inspections

Periodic post-construction inspections should be carried out at 2 to 5-year intervals depending on past experience in the climatic region of the country and past history of the cladding system on the particular structure. Such inspections should be conducted by a qualified professional whose trained eye and experience are invaluable in discerning what level of inspection is appropriate. While visual observations, including binocular scans of building envelopes, may suffice for some structures, most buildings exhibiting some measure of cladding distress will require a more detailed condition survey.

## 17.3 Causes of Distress

Since BV/CM and BV/SS cladding systems involve a variety of materials, a host of design decisions, and various construction steps under differing climatic conditions, cladding distress may be due to many causes. In essence though, distress can be viewed as being due to inadequacies in

- materials
- design
- construction
- maintenance.

In practice, distress is caused rarely by a deficiency in a single factor but rather by the combined deficiencies in several factors coming into play. For the professional involved in post-construction inspections of BV/CM and BV/SS cladding systems, it may be useful to categorize cladding distress into two key distress categories:

1. Minor distress involving more minor problems of serviceability and appearance.

2. Major distress involving problems of safety or more major problems of serviceability and appearance.

## 17.4 Signs of Minor Distress

Experience has shown that virtually all building envelopes exhibit some signs of minor distress after a few years' performance under Canadian climatic conditions. While this statement holds for all types of enclosure systems, the following comments will be directed specifically towards the BV/CM and BV/SS wall systems. Periodic inspections and maintenance procedures will ensure that minor distress problems involving serviceability and appearance are contained and do not enlarge into more major safety problems. The following listing of signs of minor distress is not meant to be exhaustive but rather illustrative of some typical situations.

Vertical cracks in brick veneer

Such cracks generally indicate either that adequate movement control joints (vertical joints) have not been provided or that significant compressive loading is present in the veneer. The former case can often be rectified by cutting control joints in alignment with the cracks, replacing badly cracked and chipped bricks, and sealing the joint with caulking on backer rod. Attempts to seal the crack simply by caulking will not affect a long term repair since the caulking cannot be made to adhere well to the cracked surfaces and the crack is often too narrow to allow penetration of the caulking material.

The second case mentioned, that of buildup of significant compressive loading in the veneer, is typically due to absence of a horizontal movement joint at shelf angles. Note that even where a caulked soft joint appears to be present, it is prudent to probe behind the caulking in cases where vertical cracking is evident: there have been too many instances where the movement space behind the caulking was mortared in. Assuming a mortared-in shelf angle condition, distress normally manifests itself by vertical cracking near wall returns and by intermittent spalling of the brick in the course above and below the shelf angle location. Should bulging of storey height panels also be indicated, major distress is present and this will be discussed in Section 17.5.

Moisture ingress at slab level

Where the reinforced concrete slab projects through to the exterior of the building envelope, moisture is able to penetrate to the building's interior all too easily. At the top of the slab the flashing must be heavily relied upon to drain any moisture from the cavity. Inevitable imperfections in the continuity of the flashing and in the required close alignment between exterior slab edge and flashing drip may create moisture paths at the floor level. The situation is worsened by differential deflections and movements between a floor slab and the cladding. These movements can create a cracked joint which readily admits water.

Differential movements and construction imperfections between slab and cladding can also permit moisture to flow along the slab's underside to cause moisture stains and drips on interior ceilings. While the common and cheapest solution is to seal affected areas by means of caulking, all such measures are short term solutions, which as part of periodic maintenance or emergency repairs, must be repeated time and time again. Experience has shown that for many Canadian climatic conditions, veneer construction with the slab projecting through to the building's exterior will require frequent maintenance measures.

Efflorescence staining

The presence of efflorescence, is invariably an indication of moisture migration. When it persists from year to year, it may adversely affect the rental or sales potential of a structure and hence will require remedial actions. What remedial actions can be taken will depend on whether the moisture is primarily the result of infiltration or exfiltration. The control of efflorescence has been dealt with in Section 14.5.

Mortar deterioration

Although today's harder cement based mortars generally display good durability, mortar deterioration may be a problem under a variety of conditions. The foremost condition involves moisture and freeze-thaw cycling. Just as the performance of concrete under moist freeze-thaw cycling is improved by air entrainment, mortar which repeatedly is subjected to such extreme conditions should be specially formulated to include an air entraining agent. While veneer walls are not normally exposed to such severe conditions, splash zones near roadways, veneer walls extending to ground level and other design features, such as poorly placed scuppers which direct moisture unto veneer surfaces, can lead to local mortar deterioration.

Brick deterioration

Although brick is a very durable material, some clay bricks as discussed in Chapter 15 some clay bricks are susceptible to freeze-thaw deterioration. Since such deterioration is dependent on the brick being saturated to a high degree, repair measures typically must address not only the replacement of damaged bricks but also the requirement for improved detailing which will prevent the accumulation of excessive amounts of moisture in the clay brick.

Weephole distress

For well functioning veneer wall systems, moisture from the cavity is vented continuously through weepholes and vents. Should weepholes be plugged or omitted altogether, moisture ingress at slab levels can readily occur. Besides flooding floor areas near external walls, damage can result to rugs and furnishings as well as to gypsum wallboard and interior finishes. Also, under cold weather conditions, freezing of water in the cavity may cause lateral pressures to be exerted on the veneer resulting in outward veneer displacement and cracking distress.

If wetting problems persist over longer periods of time, corrosion of metal components such as the bottom track of the steel stud system may ensue. Occasionally, inspection of veneered walls will reveal significant moisture staining below weepholes. This is an indication that too much moisture is entering the cavity by infiltration, exfiltration or a combination of the two. Remedial measures must address the source of the moisture to prevent premature deterioration of wall components such as ties, studs, screws, sheathing and shelf angle.

Deteriorated caulking

Even top quality, properly installed, caulking seldom has a service life exceeding 10 to 15 years when exposed to Canadian climatic conditions on the exterior of BV/CM and BV/SS wall systems. For remedial work, care should be taken to remove all deteriorated caulking and prepare surfaces according to manufacturers' instructions. Similarly, manufacturers' instructions are to be carefully followed in installing replacement caulking. In combination with a good quality caulking material, these steps will ensure a reasonable life for the re-caulked joint.

## 17.5 Signs of Major Distress

Whenever signs of minor distress are evident on the exterior or interior of a veneer clad structure, above all it is essential to investigate if otherwise hidden deficiencies endanger the safety of the veneer wall system. Such hidden deficiencies may mean that the apparently minor distress problem is really a major problem potentially endangering the safety of the public.

Perhaps the most common and potentially most dangerous cladding problem that has surfaced on highrise structures during the past 15 to 20 years has been caused by the omission of movement joints at shelf angles. This omission has led to major cladding distress in steel and especially reinforced concrete buildings; more so in the latter because differential movements here are greater due to creep and shrinkage shortening of the frame. As very substantial loads are transferred into veneers due to interaction with the frame, distress typically manifests itself in spalling of brick at shelf angle locations, vertical cracking, bulging and occasionally sudden collapse of storey height panels. While this movement joint problem has been well documented over the past few years and most designers and contractors have become aware of the vital need for the joint's presence, it is useful to focus briefly on related major deficiencies which have often contributed to the distressed condition of the veneer. Major deficiencies encountered in cladding investigations include the following:

- inadequately fastened shelf angles due to missing bolts, improperly tightened bolts, torch-cut oversized holes, undersized washers, and use of improper shimming material such as wood and bits of brick
- absence of an adequate number of ties
- corrosion of ties and other galvanized steel components such as bottom track, steel stud and screws
- cracking of CM backup due to absence of a movement joint at the top
- bulging of SS backup due to absence of a movement joint at the top or bottom
- improper installation of flashing including torn material and absence of laps
- presence of extensive mortar fins bridging the cavity and buildup of mortar droppings at the bottom of the cavity
- blocked weepholes
- inadequate bearing support of veneer on shelf angle due to large brick masonry overhangs
- wet insulation and gaps in insulation
- moisture softened gypsum wallboard
- serious lack of continuity of air and vapour barriers.

When listing these major deficiencies, it should be kept in mind that some stem from the original construction while others are the result of the cladding system's distress over a period of time. Because so many potentially major deficiencies are hidden at the outset of an enclosure investigation, it is vital that any investigation be thorough enough to establish the true extent of distress and the potential for unsafe conditions or costly future repairs.

## 17.6 Performance of Cladding Systems

Besides the BV/CM and BV/SS systems, other modern cladding systems used in Canada include materials such as precast concrete, stone, steel and aluminum. Mention of the wider range of cladding systems is made here because the discussion in the last two sections may have left the impression that the BV/CM and BV/SS systems are prone to performance failure. That would be an erroneous impression indeed: to the authors' knowledge, both systems have performed satisfactorily and it is in fact that satisfactory performance at economic cost which leads to their continued widespread use. Examples of minor and major distress can be cited for all cladding systems, be they those dealt with in this publication or others. Canada-wide environmental conditions acting on cladding systems differ according to regions, yet in a broad sense most Canadian conditions are relatively harsh. For the satisfactory performance of the BV/CM and BV/SS wall systems over the long term, it is important that all aspects of material selection, design, construction, inspection and maintenance are carried out with due care for the harsh Canadian environmental conditions.

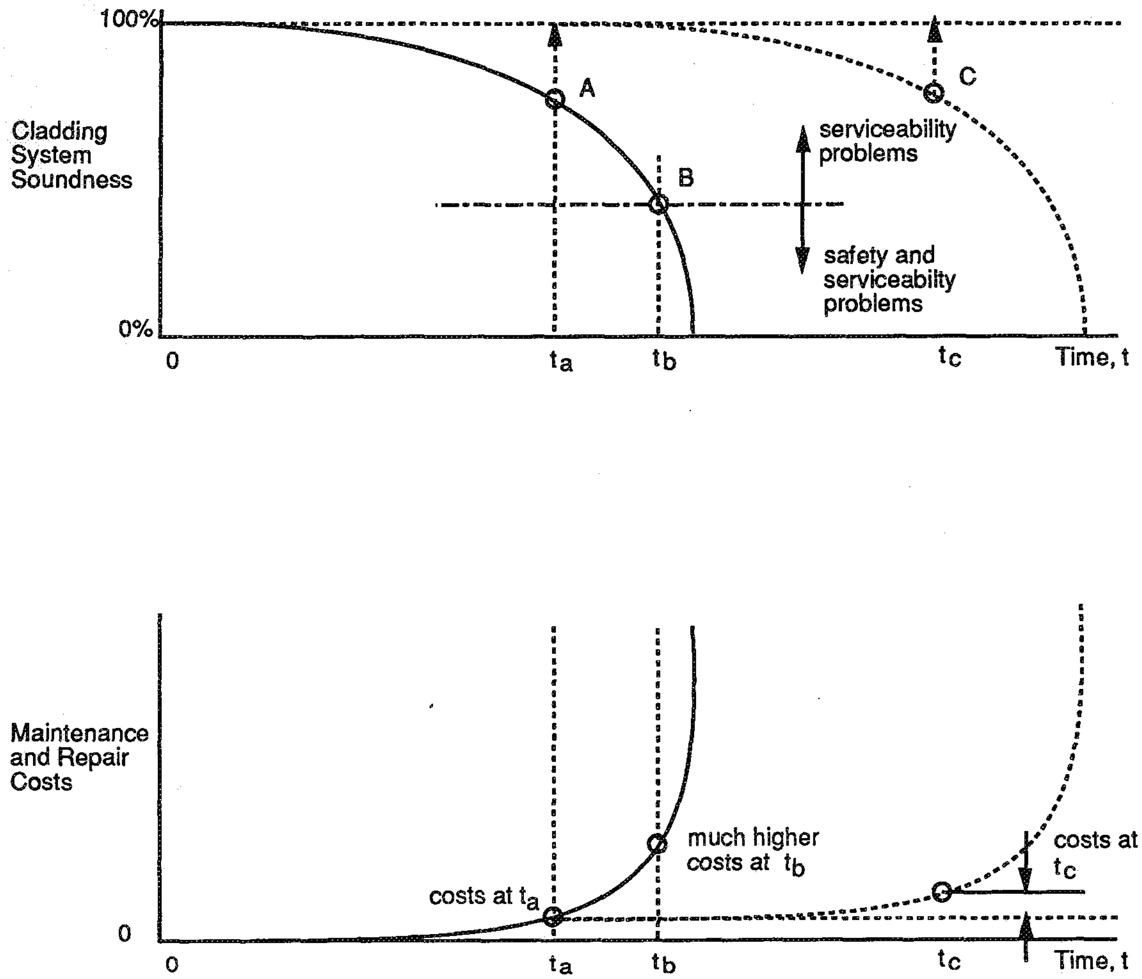



Fig. 17.1 Schematic illustration of soundness and maintenance/repair costs versus time for a cladding system

Chapter 18

# A SUMMARY OF

# STRENGTH AND STIFFNESS CHARACTERISTICS OF STEEL STUD BACKUP WALLS DESIGNED TO SUPPORT BRICK VENEER

by

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## 18.1 INTRODUCTION

This report contains research information on the strength and stiffness behaviour of steel stud walls constructed to act as the backup for brick veneer facing on buildings. The research reported comprises Part 1 of a five part laboratory test program at McMaster University sponsored by the Project Implementation Division of Canada Mortgage and Housing Corporation. While information in this report is relevant for other uses of steel stud construction, the primary objective of this investigation was to focus on those aspects directly relating to the performance of the combined brick veneer/steel stud (BV/SS) wall system.

A brief synopsis of the most recent laboratory research projects dealing with with the behaviour of the backup wall assembly follows.

### 1) Clemson Study[1]

This study was co-sponsored by the Brick Institute of America (B.I.A.) and the Metal Lath/Steel Stud Framing Association and was conducted at Clemson University under the direction of R.H. Brown. A first phase provided documentation of the performance of two different types of metal ties used to fasten brick veneer to steel studs. In a second phase, six BV/SS wall panels were tested under lateral load. Three walls were tested under positive lateral load and three under negative lateral load using an air bag to provide a uniformly distributed load. Two 3-5/8 inch (92 mm) deep, 20 gauge cold formed channel sections, spaced 24 inches (600 mm) apart were used for each backup wall panel. One line of 16 gauge bridging was inserted into a web cutout hole at midspan of the steel stud. This row of bridging was fastened to each stud with clip angles and screws. All panels were identically sheathed on the interior and exterior with 0.5 inch (12.5 mm) thick gypsum board. The sheathing was jointed horizontally at mid height of the wall and was fastened with Number 6-DG screws. Some of the key conclusions reached were:

1. Flexural cracking of brick veneer was improbable under twice the design lateral wind load for an L/360 design criteria for deflection.
2. Composite action between the steel stud and the gypsum board sheathing, although present, was not significant.
3. Tie forces were not uniform.
4. Allowable stress in metal studs should not be exceeded.

### 2) University of Alberta Study[16]

This study was conducted in 1985 at the University of Alberta under the direction of M.A. Hatzinikolas et al., and was co-sponsored by Dow Chemical Canada Incorporated and the Prairie Masonry Research Institute. The thirty-two BV/SS wall specimens were approximately 1200 mm wide and the steel studs spanned 3 metres while the brick veneer was 3.2 metres in height.

Nine steel stud wall panels were also built and tested. Eight of the test panels had the compressive face of the steel studs sheathed with either gypsum board or Styrofoam S.M. insulation. The tension face of all eight panels were sheathed with gypsum board. The panels were additionally braced with two lines of channel bridging welded to the studs at the web cutout holes. The last panel was not provided with any type of exterior sheathing. All panels were tested under third point loading. It is interesting to note that this effectively placed the lcoation of each point load at approximately the same location as each line of bridging.

Some of the more important findings and conclusions included the following:

1. Brick veneer cracking will result if design practice allows the midspan deflection of the steel studs to be L/360.

2. Gypsum board provides more lateral bracing than polystyrene insulation

3) **Murden, J.A. (M.Eng. Thesis)[24]**

An experimental investigation into the out-of-plane stiffness of steel stud backup walls subjected to cyclic loading was carried out at Clemson University under the guidance of R.H. Brown.

The experimental program included cyclic testing of eleven backup panels in which seven were fabricated with 20 gauge studs and track and four with 16 gauge studs and track. All the studs were 3-5/8 inches in depth and were approximately 7 feet 10 inches long. Each panel was sheathed with 0.5 inch thick exterior and interior gypsum board. The sheathing was installed in a continuous fashion or with a butt joint at midspan. The sheathing was attached to the flanges of the studs with 1 inch long Number 8 drywall screws. The panels were loaded in an alternating manner with a single midspan load which caused a midspan panel deflection of slightly less than L/360 in each direction. Each panel underwent 5000 loading cycle. Some important findings were:

1. The composite action between the gypsum board and the steel studs was independent on the spacing of the drywall fasteners and the facing orientation of the drywall sheaths on the steel studs. The amount of composite action was also quite variable and very sensitive to the care taken during installation of the sheathing.

2. Cyclic loading decreased the amount of composite action very rapidly.

## 18.2 CURRENT DESIGN CRITERIA

In Canada there are no specific codes which govern the overall design, construction and inspection of a BV/SS curtain wall systems. However, CAN3-S304-M84[22] governs the design of masonry structures and CAN3-S136-M84[6] covers the design of cold form steel structural members. In the masonry code, brick veneer is defined as a non-load bearing facing which is attached to a structural backing and is not relied on to act with the structural backing to resist any lateral load. For the more traditional brick veneer and masonry block backup, this assumption is reasonable since the stiffer backup wall resists the majority of the load. However, when a brick veneer with a steel stud backup wall is used, the brick veneer is much stiffer than the backup and will resist a much greater portion of the lateral load, at least until it cracks. If the brick veneer is assumed to be a structural element, then Table 3 of the masonry code limits the allowable tensile stresses normal to the mortar bed joint to 0.26 MPa for Type S mortar. However BIA Technical Note 28B 4 states that the brick veneer/steel stud wall system should not be designed using the allowable flexural tensile stresses as contained in the structural design standards for masonry. It was suggested that the design of the wall system be empirically based on past observations and experience. This has led the B.I.A. to recommend a maximum deflection limitation of L/600 to L/720 when the steel stud backup alone is considered to resist the full unfactored lateral design load. It was concluded that this will ensure the necessary stiffness for satisfactory performance. An additional requirement was that the steel studs must be securely sheathed on both sides. Also, only Type S mortar was recommended for use in brick veneer walls at locations where wind loads are expected to exceed 1.2 KN/m$^2$.

Others[23] contend that the L/360 deflection limitation for the steel stud backup acting alone, is acceptable. However, they also stated that the L/360 design critieria should be limited to the parameters of the Clemson test assembly and that for other applications, some judgement by the designer is required.

Based on the above, steel stud manufacturers have developed wind load design tables for their products. These tables are usually based on an L/360 or L/600 deflection criteria under unfactored design wind loads. Regarding strength requirements for the steel studs, these tables are usually based on the assumption that adequate stud bracing is provided. It is then left to the designer to decide what bracing is required. However the current cold formed design standard[6] does not have any specific guidance for perforated studs.

## 18.3 GOALS AND OBJECTIVES

An experimental test program was be undertaken to document and evaluate the strength and stiffness characteristics of various components of the steel stud backup wall assembly. To accomplish this, the laboratory test program was planned with the objective of achieving the following goals:

1. Documentation of the bending, torsional and web crippling strengths as well as the deformational behaviour of steel studs.
2. Provision of data on strength, stiffness and construction features for steel stud to track, top and bottom connection details.
3. Evaluation of the effectiveness of various currently used types of bridging and bridging connections.
4. Determination of the bracing capacity of gypsum board as well as other sheathing materials.
5. Observation of effects of cyclic loading and wetted gypsum board on the stiffness of the backup wall.

## 18.4 COLD FORMED STEEL STUD-TRACK CONNECTION TESTS

### 18.4.1 Design And Set-Up Of Experiments

A simple test set-up was devised to investigate and document the strength and behaviour of various steel stud to track connection details. The horizontal translation of the steel stud end was expected to be due mainly to the transfer of lateral load from the end of the steel stud to the supporting steel track. In order to simulate this type of action, an experiment was designed which isolated the transfer of lateral load. To accomplish this, a short section of steel stud was fastened to a length of track, as shown in Figure 18.1, using a specified fastening detail. The track was fastened to a concrete beam which was used to simulate a typical floor slab. In order to minimize the influence of the flexural deflection of the steel stud, the free end of the short length was supported by a load cell. Use of the load cell made the specimen statically determinate and provided a means of obtaining the lateral force at the track. The test apparatus was designed to fit into an hydraulic test machine as shown in Figure 18.2. This facilitated the application of the total lateral load to the steel stud.

The above experiment has the advantage of isolating the effect of shear transfer of one stud on a short section of track. However, in a typical steel stud backup wall, steel studs are usually spaced 300 mm to 400mm on center. Therefore, it is possible that nearby studs might influence the translational characteristics of the stud to track connection under observation. This can be investigated by fastening two short lengths of steel stud to the track, and loading both studs simultaneously. Comparison of these results with those for single studs should

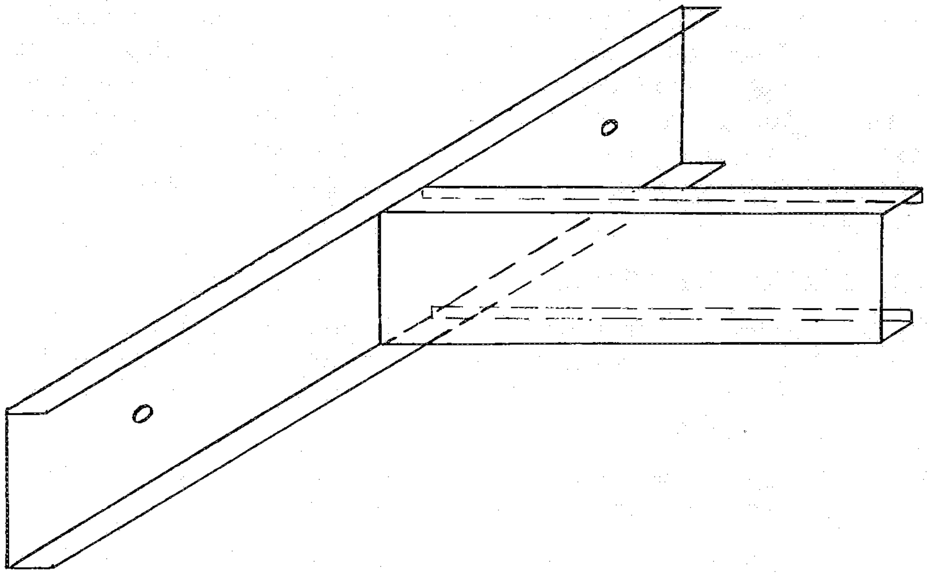

Figure 18.1 **Steel Stud to Track Connection Specimen**

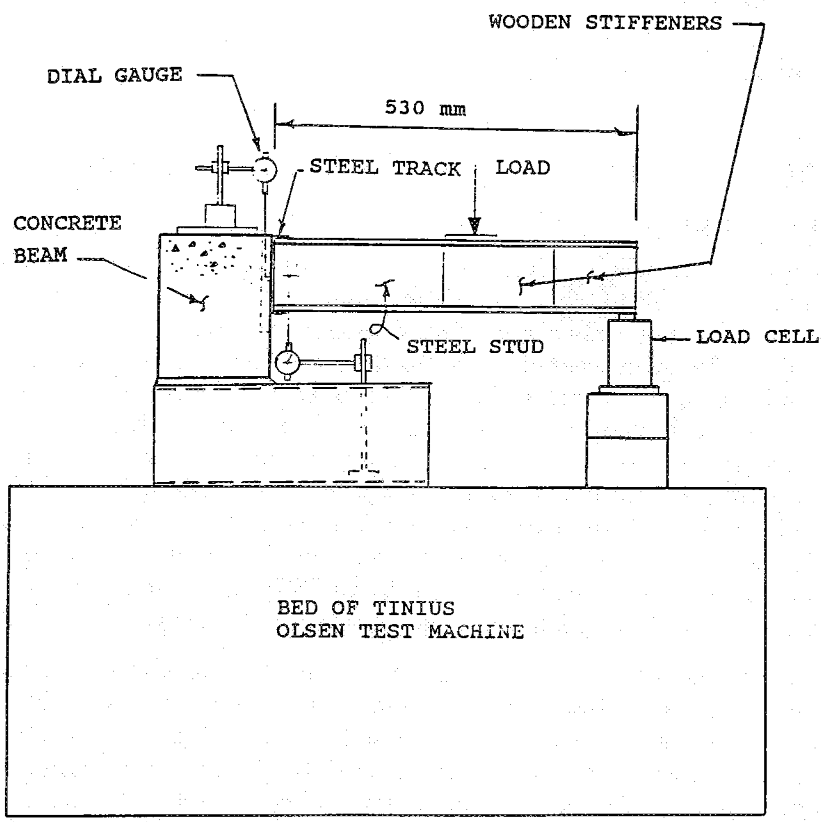


Figure 18.2 **Experimental Set Up Used for Steel Stud to Track Connection Tests**

indicate whether the translational characteristics of a particular connection is local in nature or if it is significantly influenced by the action of nearby studs.

Preliminary investigation showed that the rotational restraint of the track at the stud to track connection was not significant and therefore no tests were done to provide moment-rotation relationships.

To investigate the connection strength at the bearing end of a cold formed steel stud at the stud track interface, the experiment was designed to allow failure to occur at the connection and not prematurely under the load applied to the top flange of the steel stud. This was accomplished by stiffening the steel stud with wooden blocks at locations other than at the stud to track connection. This will be discussed in more detail later.

Various types of track anchors were also investigated. In the majority of tests, each track was fastened to the concrete beam using expansion anchors. However, in practice nail anchors are generally used. Each nail anchor is fired from a specially designed gun which uses an explosive charge as the driving force to drive the nail through the steel track and into the supporting concrete slab or support member. For the strength characteristics of this type of anchor be investigated, the last eleven tests in this phase used nail anchors to fasten the steel tracks to the concrete beam.

### 18.4.2 Preparation And Fabrication Of Test Specimens

The test specimens were fabricated from cold formed track and channel shaped members having dimensional properties shown in Figure 18.3 and listed in Table 18.1. Typical specimen cross-sections used shown in Figure 18.4. The 0.53 meter long steel stud specimens were cut from 2.59 meter long cold formed steel members using a band saw. The tracks were similarly cut into 1.22 metre lengths from 3.048 metre long sections. Only sections of steel stud and track free from visible damage were used to fabricate the test specimens. Each short length of steel stud was attached at the center of a steel track specimen.

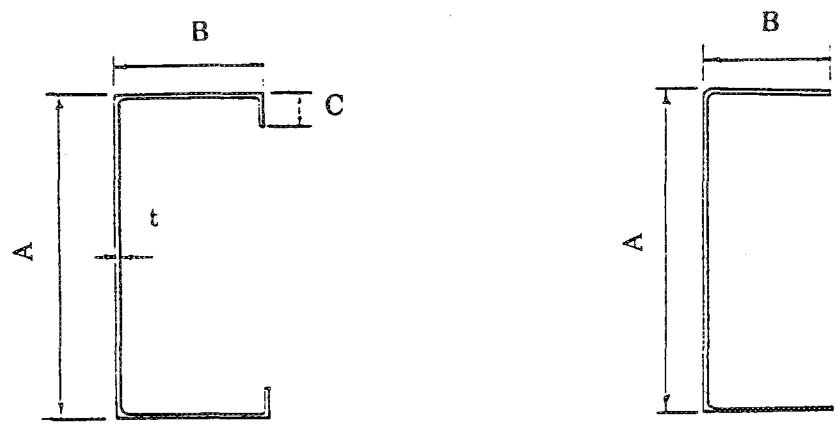


FIGURE 18.3 TYPICAL CROSS SECTIONS USED IN TEST PROGRAM

TABLE 18.1

STEEL STUD AND TRACK SECTION GEOMETRIC PROPERTIES

| Stud Type | Dimensions (mm) A | B | C | Thickness t (mm) |
|---|---|---|---|---|
| 20 ga. | 92.08 | 34.93 | 9.53 | 0.91 |
| 20 ga. | 152.40 | 34.93 | 9.53 | 0.91 |
| 18 ga. | 92.08 | 41.28 | 12.7 | 1.22 |
| 18 ga. | 152.40 | 41.28 | 12.7 | 1.22 |

| Track Type | Dimensions (mm) A | B | C | Thickness t (mm) |
|---|---|---|---|---|
| 20 ga. | 92.58 | 33.35 | – | 0.91 |
| 20 ga. | 152.90 | 33.35 | – | 0.91 |
| 18 ga. | 92.58 | 33.35 | – | 1.22 |
| 18 ga. | 152.58 | 33.35 | – | 1.22 |
| 14 ga. | 92.58 | 33.35 | – | 1.90 |
| 20 ga. | 92.58 | 63.50 | – | 0.91 |
| 20 ga. | 152.58 | 63.50 | – | 0.91 |

Various connection details were used to attach the steel stud to a track. Each type of connection tested is described in the next section. In order to fasten the track quickly to the concrete beam in the test rig, two 10 mm diameter holes were drilled into the web of the track at locations shown in Figure 18.4a). This allowed the track to be fastened to the concrete beam using two 8 mm diameter threaded rods. The rods had been previously screwed into expansion type anchors which had been set in holes drilled in the concrete beam. This provided a reasonable method of attachment were no damage or displacement occurred in either the concrete beam or the anchor bolts through many test repetitions.

For the test specimens which used screws to connect the end of the stud to the track, an electric screw gun was used to drive number 6 pan head self-drilling screws. The steel stud was first aligned in the track and secured in this position with locking pliers. The gun was then used to drive the self-tapping screws through the connecting pieces.

Table 18.2 contains a summary of stud-track connection details.

Figure 18.5 is a photograph of a typical test set-up for a 90 mm, 20 gauge, Series D1 specimen. Normally each specimen was loaded in 500 newton increments, but in some tests, 250 newton increments were used. The load head was lowered at a rate of 0.15 inches per minute. This was increased to approximately 0.20 inches per minute as the loading approached the ultimate value. At the end of each load increment, deflection readings were recorded. Failure was defined at the point where the load dropped off significantly with increased displacement.

### 18.4.3 Results of Steel Stud to Track Connection Tests

In most test series it was noted that the load-displacement relationships were linear for loads up to approximately 75 to 80 percent of ultimate. Figures 18.6a), b) and 18.7a), b) represent this linear range. Since the stud end translation should usually be limited to a value no greater than 2 mm, the mean load at this displacement was included in column eight of Table 18.3.

TABLE 18.2

STUD-TRACK CONNECTION DETAILS

| Series | Connection† | Gap |
|---|---|---|
| D1 | T, C | minimum |
| D2 | T, C | 12 mm |
| D3 | T | minimum |
| D4 | C | minimum |
| D5 | T | 12 mm |
| D6[1] | T, C | minimum |
| D7[2] | clip angle | |
| D8[3] | flexible angle | 12 mm |
| D9[4] | box track | 12 mm |
| D10[5] | T, C | minimum |
| D11[6] | T, C | minimum |
| D12 | T, C, nested | 12 mm |
| D13 | welded | minimum |
| D14[7] | T, C | |

1 stud attached at end of track, see Figure 18.4b)
2 see Figure 18.4c)
3 see Figure 18.4d)
4 see Figure 18.4e)
5 20 gauge stud/14 gauge track
6 20 gauge stud/18 gauge track
7 two stud connection, see Figure 18.4h)

† refer to Figure 18.4a) except as noted
T = Tension Flange screw
C = Compression Flange screw

The average maximum lateral load for each test series at the track to stud connection is listed in column five of Table 18.3. Lateral load values were obtained by subtracting the value recorded by the load cell which supported the free end of the stud from the total lateral load applied to the top flange by the Tinius Olsen Machine. Column seven is the mean yield load for each individual test series. This value basically defines the load level that marks the beginning of large permanent deformations at the stud to track connection and as such is a useful limit. The mean yield value was obtained graphically and is only approximate since a different observer might select a slightly different value.

### 18.4.4 Summary of Stud-Track Connection Tests

In nearly all of the steel stud to track connection tests, the track anchors were spaced approximately 900 mm on centre. For this anchor spacing, the track sections and expansion type anchors used were found to perform adequately. However in some preliminary tests the track anchors were spaced at 1500 mm on centre and it was found that a greater track deflection occurred and in some cases the top flange of the track buckled before the stud failed by web crippling. Since the installation of the track anchors is a field operation and proper track anchorage is required to control the out of plane deflections of the steel stud backup wall assembly it was concluded that an anchor spacing of 800 mm on centre or less should be used regardless of the type of anchors used.

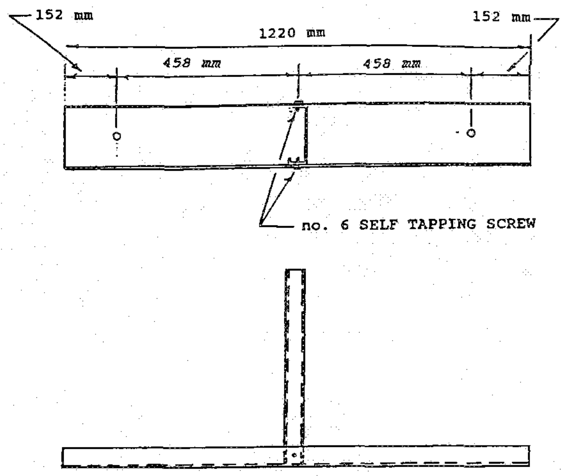


**Figure 18.4a)**

**Typical Specimen Used for Series D1**

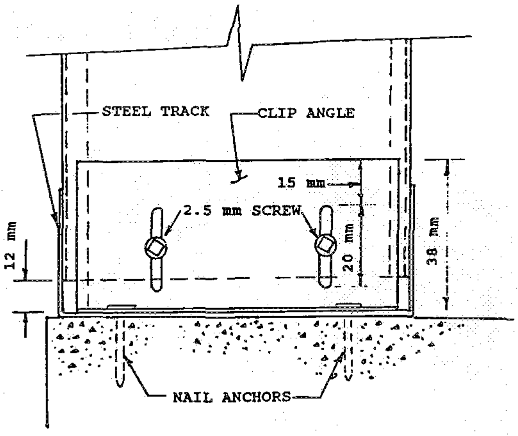


**Figure 18.4c)**

**Typical Clip Angle Connection Detail Used for Series D7**

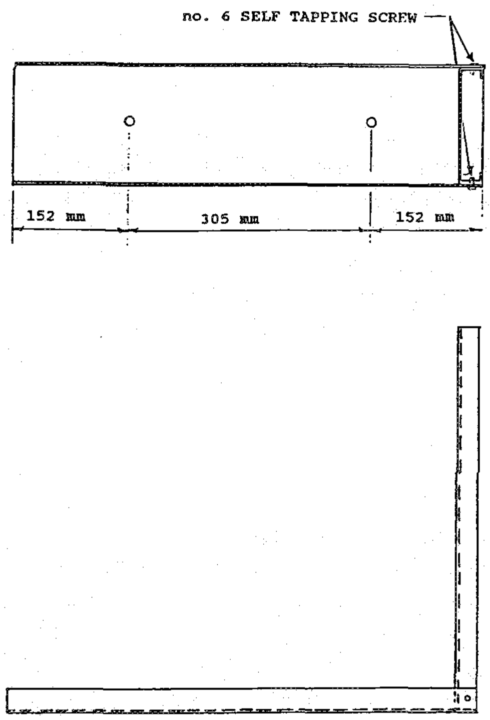


**Figure 18.4b)**

**Typical Specimen Used for Series D6**

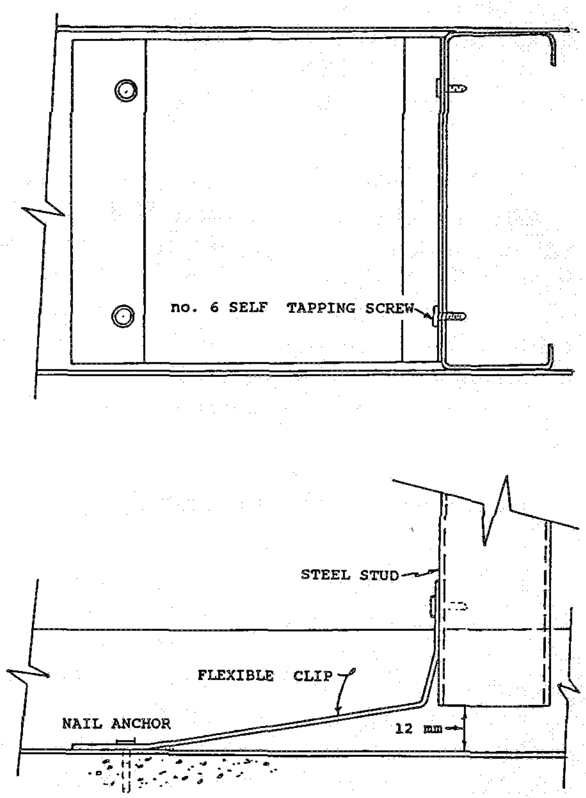


**Figure 18.4d)**

***Typical Flexible Clip Connection Detail Used for Series D8***

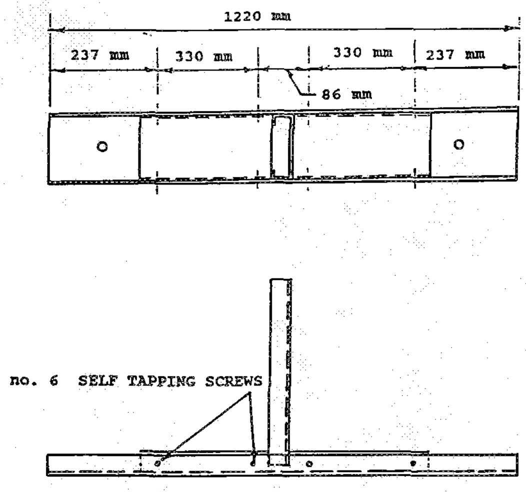


Figure 18.4e)

Typical Box Track Detail Used for Series D9

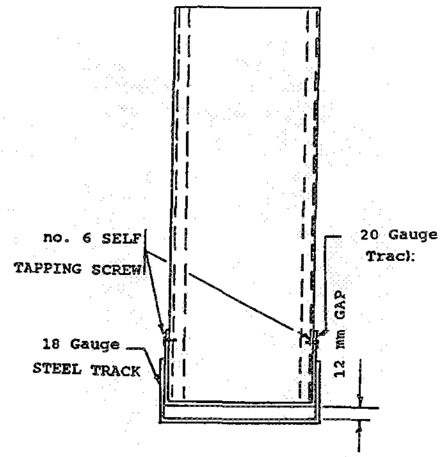


Figure 18.4f)

Typical Nested Track Detail Used for Series D12

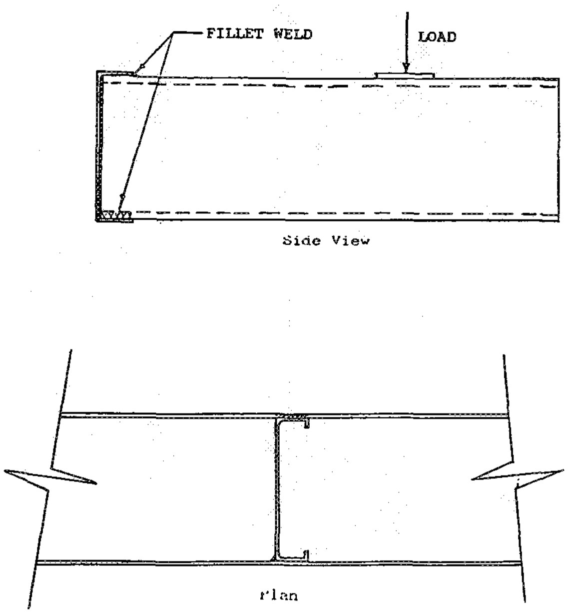


Figure 18.4g)

Typical Welded Stud to Track Connection Detail Used for Series D13

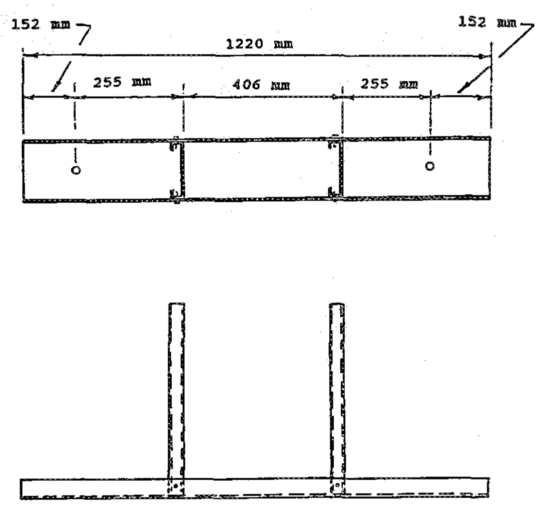


Figure 18.4h)

Two Stud Specimen Used for Series D14

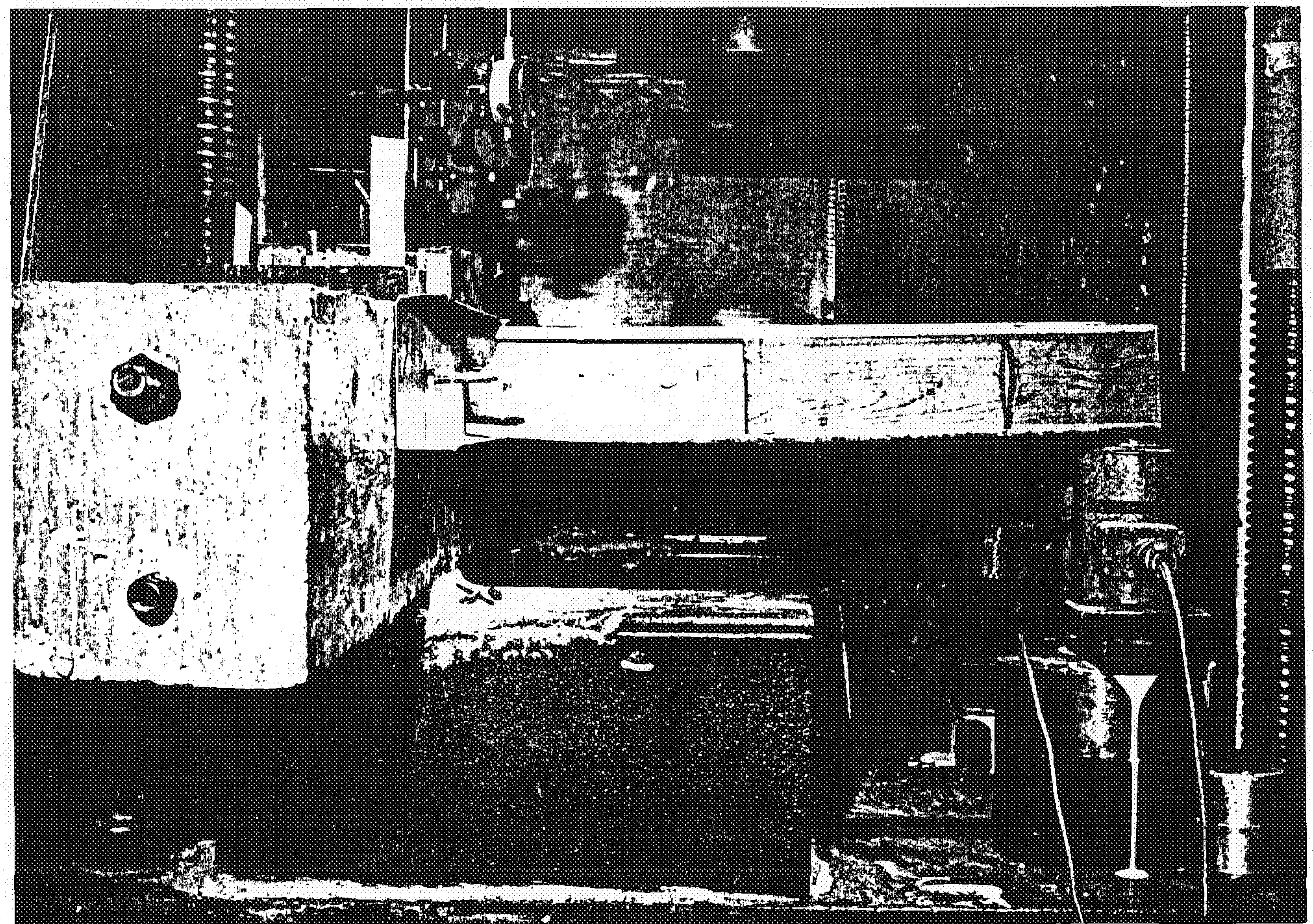

Figure 18.5 Photograph of 20 Gauge, 90 mm Stud to Track Test Set Up - Series D1

TABLE 18.3

SUMMARY OF STUD-TRACK FAILURE LOADS

| Stud Gauge | Size | Connection Series | No. of Specimens | Ultimate Load (kN) | COV (°6) | Yield Load (kN) | Load @ 2 mm Displ. (kN) |
|---|---|---|---|---|---|---|---|
| 20 | 90 | D1 | 9 | 2.82 | 7.1 | 2.00 | 1.11 |
| 18 | 90 | D1 | 5 | 4.70 | 5.1 | 3.70 | 1.93 |
| 20 | 150 | D1 | 6 | 2.65 | 5.9 | 1.90 | 1.25 |
| 18 | 150 | D1 | 6 | 4.43 | 5.6 | 3.30 | 2.03 |
| 20 | 90 | D2 | 4 | 2.81 | 6.1 | * | 0.49 |
| 20 | 150 | D2 | 4 | 2.50 | 4.6 | * | 0.47 |
| 18 | 90 | D2 | 4 | 4.65 | 3.6 | 2.80 | 0.96 |
| 18 | 150 | D2 | 3 | 4.09 | 4.5 | 3.40 | 0.98 |
| 20 | 90 | D3 | 3 | 2.63 | 8.0 | – | – |
| 18 | 90 | D3 | 4 | 4.37 | 4.2 | 2.90 | 1.40 |
| 20 | 150 | D3 | 3 | 2.30 | 1.6 | 1.90 | 1.07 |
| 18 | 150 | D3 | 3 | 3.95 | 5.5 | 3.00 | 1.56 |
| 18 | 90 | D4 | 4 | 4.38 | 8.8 | 3.30 | 1.80 |
| 20 | 90 | D5 | 4 | 2.35 | 10.1 | – | – |
| 20 | 150 | D6 | 2 | 2.39 | – | 1.90 | 1.17 |
| 18 | 90 | D7 | 2 | 4.58 | – | 3.00 | 3.65 |
| 18 | 90 | D8 | 3 | 4.82 | 7.7 | 3.00 | 5.53 |
| 18 | 90 | D9 | 2 | 4.27 | – | 2.90 | 0.78 |
| 20 | 90 | D10 | 3 | 3.07 | 1.8 | 2.50 | 2.81 |
| 20 | 150 | D11 | 3 | 3.17 | 5.2 | 1.90 | 1.53 |
| 20 | 90 | D12 | 3 | 2.50 | 6.8 | * | – |
| 18 | 150 | D13 | 3 | 5.134 | 11.9 | * | 8.33 |
| 18 | 90 | D13 | 3 | 5.15 | 9.3 | * | 4.28 |
| 18 | 90 | D13 | 3 | 4.81 | 0.8 | * | 3.69 |

When the results of the two stud tests, Series D14, were compared to the results of Series D1, it was concluded that the out-of-plane deformations of each stud was not influenced to a great extent by the actions of nearby studs. This is due to the fact that the deformations at the track to stud connections are mainly local in nature .

The test results indicated that the two screw steel stud to track connection detail provided a much stiffer connection detail than the one screw connection . However the welded and the clip angle type of connection details were found to provide the stiffest type of connection detail.

## 18.5 STEEL STUD BACKUP WALL PANEL TESTS

### 18.5.1 General

Steel stud wall panels were tested to assess the effects of various design or construction practices on the behaviour of steel stud backup wall systems intended to support brick veneer.

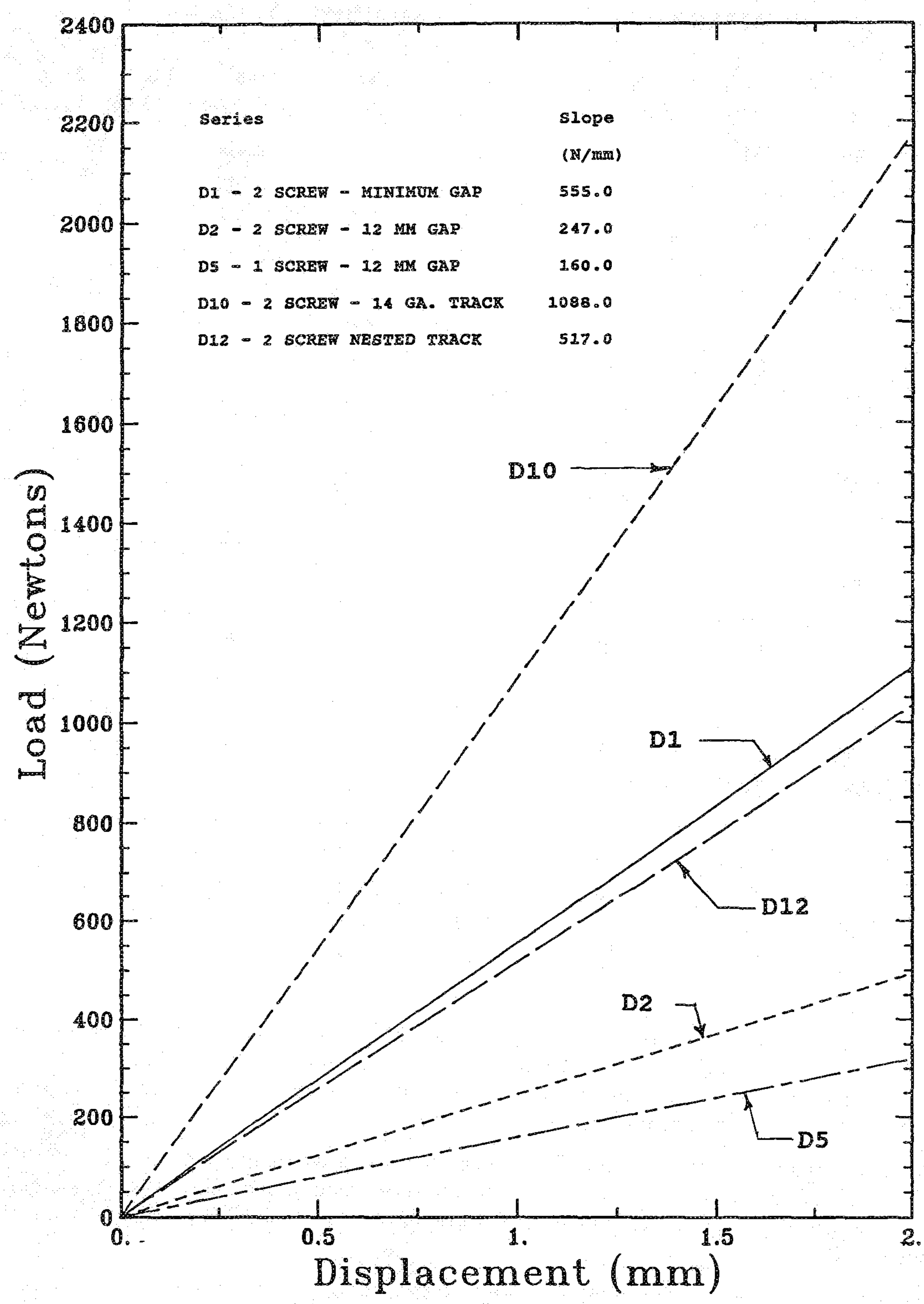


Figure 18.6a) Load Versus Displacement Summary 20 Gauge, 90 mm Deep Steel Stud to Track Connection Tests

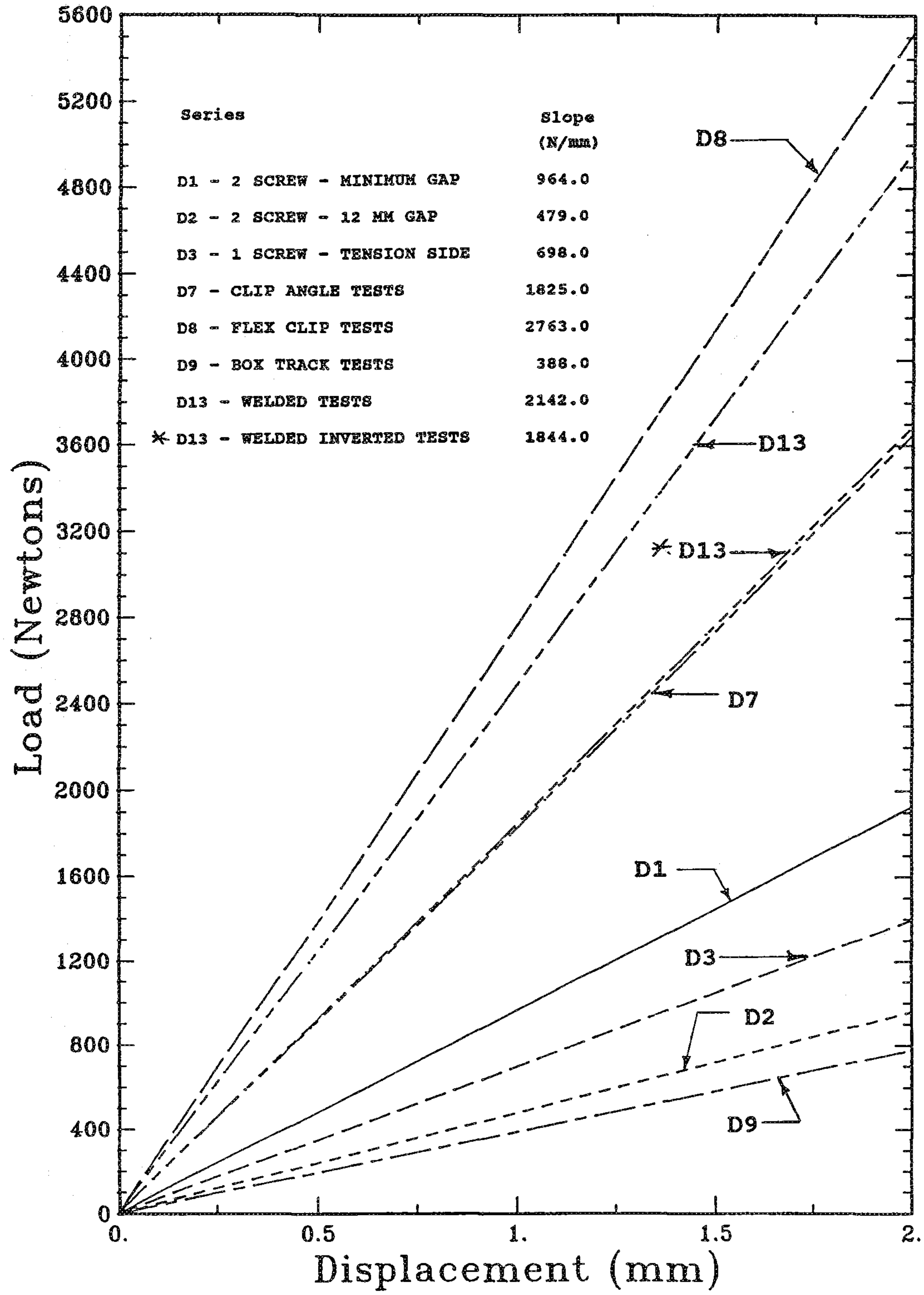


Figure 18.6b) Load Versus Displacement Summary 18 Gauge, 90 mm Deep Steel Stud to Track Connection Tests

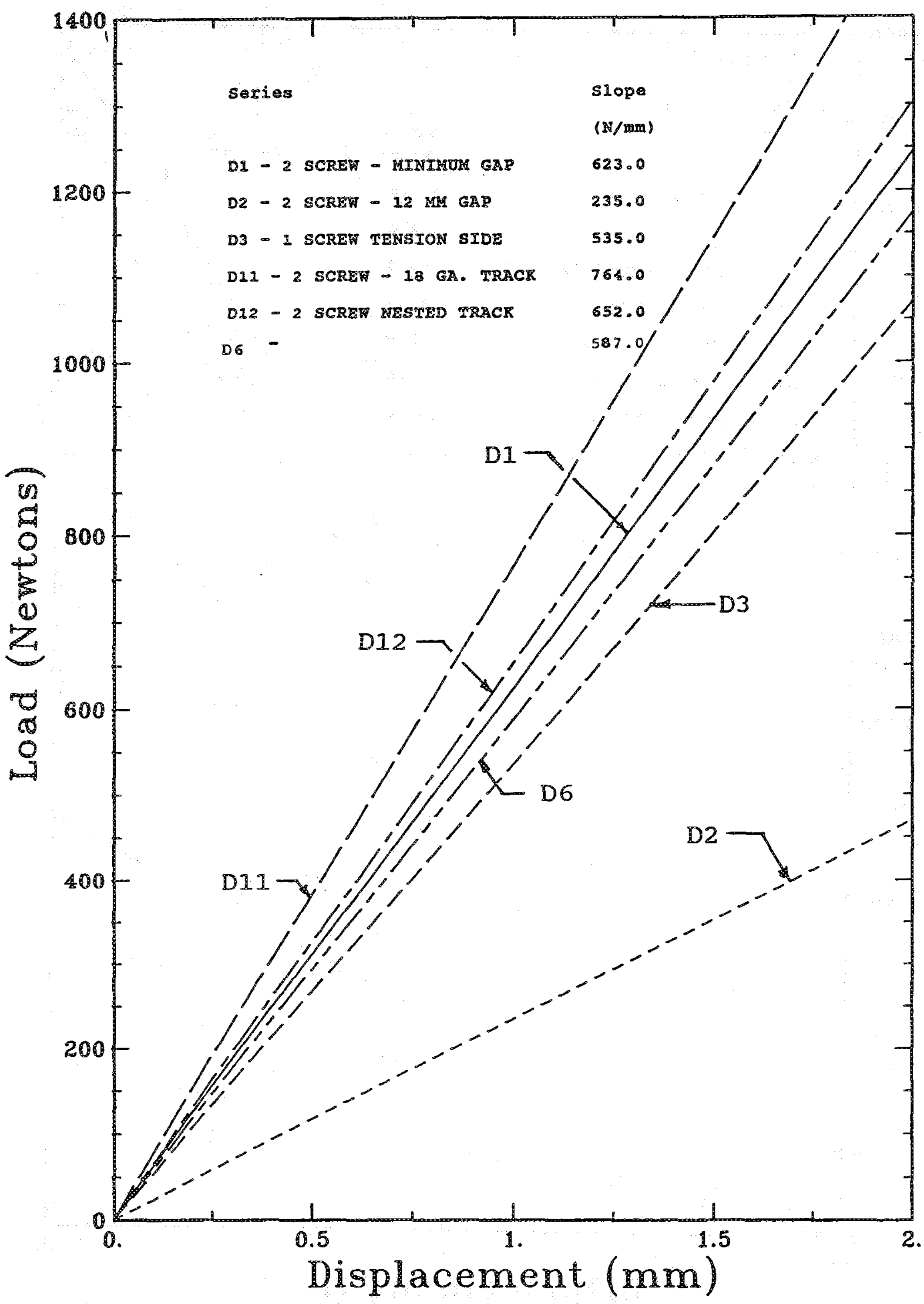


Figure 18.7a) Load Versus Displacement Summary 20 Gauge, 150 mm Deep Steel Stud to Track Connection Tests

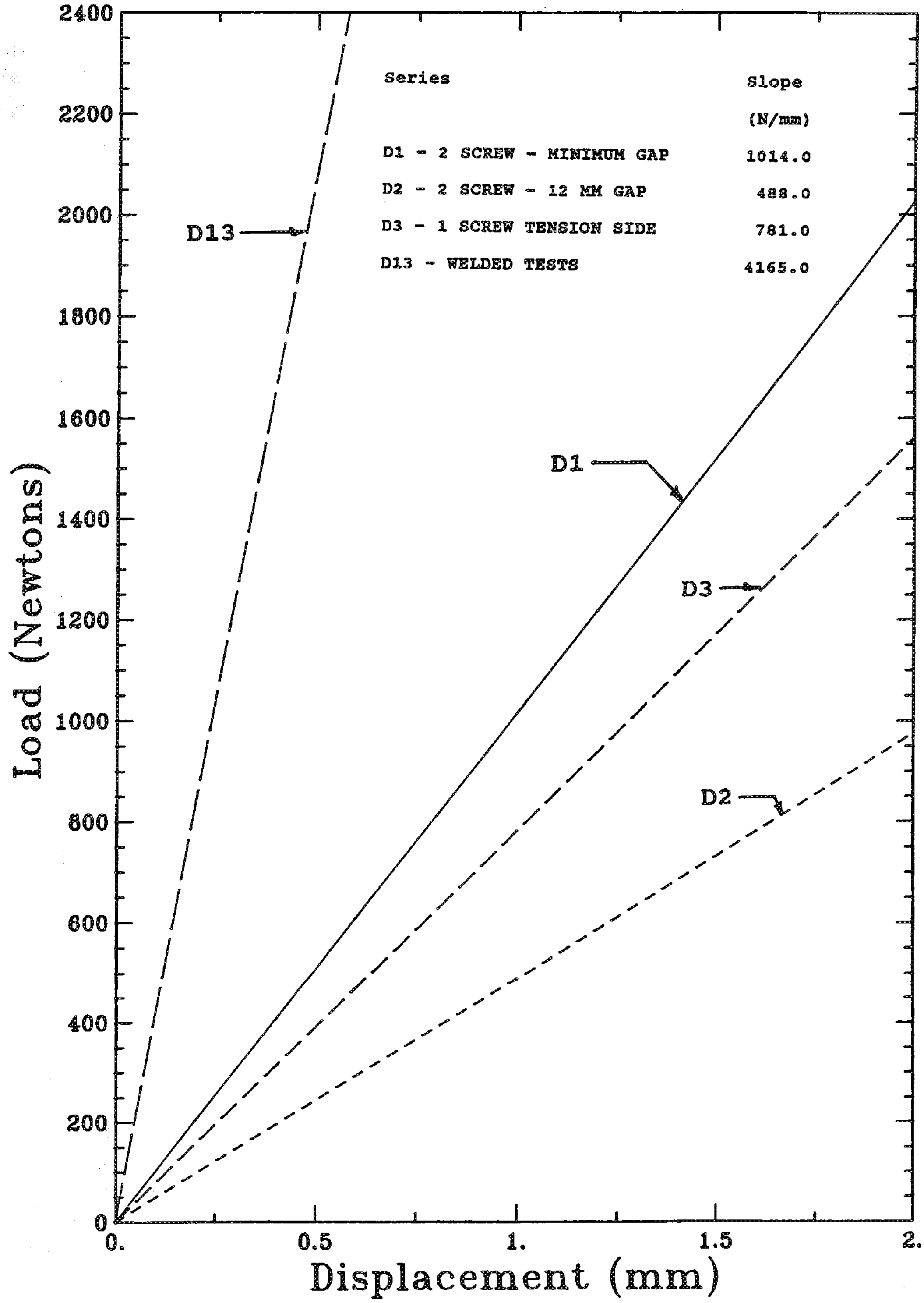


Figure 18.7b) Load Versus Displacement Summary 18 Gauge, 150 mm Deep Steel Stud to Track Connection Tests

Separate tests of steel stud wall assemblies were designed to investigate specific aspects. Selection of test conditions, design of test specimens, design and fabrication of tests apparatus, test procedure and the test results are included in this chapter. In addition general observations and conclusions arising from these tests results are provided.

The objective of this part of the experimental research program was to document the influence of various factors on the strength and stiffness of the complete steel stud backup wall assembly. Therefore it was necessary to design a test program which would include information on:

- influence of realistic support conditions
- effectiveness of internal (through the web) versus surface bridging to prevent premature torsional-flexural buckling
- influence of connection conditions and spacing of bridging
- interaction of gauge of steel stud with other factors
- effectiveness of sheathing to provide composite action and/or torsional bracing
- effect of repeated loading
- influence of the type and method of connection of sheathing.

### 18.5.2 Design of the Wall Test Specimens

The experiment had to be designed to provide full information while not over complicating either the fabrication or the the test procedures. To accomplish this and still have a realistic experimental model, consideration was given to the following :

1) In plane height of backup wall assembly,
2) Spacing and number of studs used,
3) Type and number of loads on each stud,
4) Continuity of stud bracing,
5) Support track anchor spacing, and
6) Type of wall studs.

The in-plane height of the backup wall assembly was chosen to be 2.59 metres which is in the range commonly used for residential construction. The wall studs consisted of 90 mm deep lipped channels which are normally used for this range of backup wall heights. In all the tests, the commonly specified stud spacing of 406 mm on centre was used.

Each steel stud in the backup wall panel was symmetrically loaded with two equal concentrated loads located approximately at the quarter points of the span. This type of loading was chosen since the maximum moment between the loads was approximately equal to the maximum moment for uniformly distributed load. Also, two point loading on the compression flanges of the studs was used to simulate the most severe concentrated loading condition for lateral wind load on the backup wall. In general the transfer of lateral loads to the backup wall through brick ties or by direct air pressure represents a less severe loading condition for the stud.

Four stud backup wall panels were required to investigate the strength and deformation characteristics of backup walls braced at discrete locations or with studs braced with sheathing and discrete bracing. This length was necessary to allow the continuity of bracing to be reasonably well modelled for the interior studs. Therefore this test was designed to allow the two interior studs to fail first. This was accomplished by providing additional lines of bracing for the exterior studs. The discrete bracing consisted of various types of steel bridging and the location and number of rows of bridging were varied in the test program.

The loading arrangement used in the four stud backup wall tests was designed to allow independent rotation of each stud. This minimized the potential for unintentional lateral bracing at the load points which might otherwise have been provided by the loading arrangement.

Two stud test specimens were used to test steel studs with gypsum board sheathing on both the interior and exterior faces of the wall. The use of two stud wall panels was sufficient since it was expected that the sheathing would prevent buckling of the studs. For these tests, the continuity of bridging was not considered. Each panel was cyclically loaded in order to investigate whether any additional stiffness of the backup wall panel achieved through composite action would be sustained after repeated cycles of loading. Two stud backup wall panels were also used for unbraced stud tests.

Forty-five steel stud wall panel specimens were fabricated and tested. These tests were divided into seven tests series. For simplicity, all steel stud backup wall panels were fabricated in a horizontal position. Typical types of studs used in the fabrication are shown in Figure 18.8. These steel studs had prepunched 102 mm by 38 mm web cutout holes located as shown. In some tests, steel bridging was inserted through some of these holes to provide bracing of the steel studs at these locations.

The steel stud backup wall specimens were tested horizontally in the test frame where the top and bottom of the frame, representing the floors of a building, consisted of 200 mm deep by 150 mm wide by 1.83 metres long reinforced concrete beam, while the sides consisted of 300 mm steel channels. Load was applied by hydraulic jack attached to a loading frame as shown in Figure 18.9.

During the backup wall tests, metric dial gauges and linear potentiometric displacement transducers (L.P.D.T.) were used to measure the out-of-plane deflections. Also, rotation measurements were taken in Series 1 through 3 with the aid of a protractor and a plumb-bob.

### 18.5.3 Results

The following is an overview of the results for the wall panel tests. In general, it was observed that the failure of a steel stud backup wall panel was initiated when one or more studs started to twist significantly. Failure of a stud was always observed to occur in the region around one or more web cutout holes. Visual examination of the panels immediately after testing indicated that no significant track flange deformation and/or stud web crippling occurred at the stud to track connections.

The failure loads for Series 1 through 7 are listed in Table 18.4. Each steel stud in a backup wall panel was loaded with two equal concentrated loads. For each test the value listed in Table 18.4 is the total load on one steel stud at failure. For comparison purposes, the results of each series the failure loads obtained for the 20 gauge backup wall specimens were divided by the average failure load obtained from the results of 20 gauge beam tests. The failure loads for the 18 gauge backup wall specimens were treated in a similar manner using the results of beam tests.

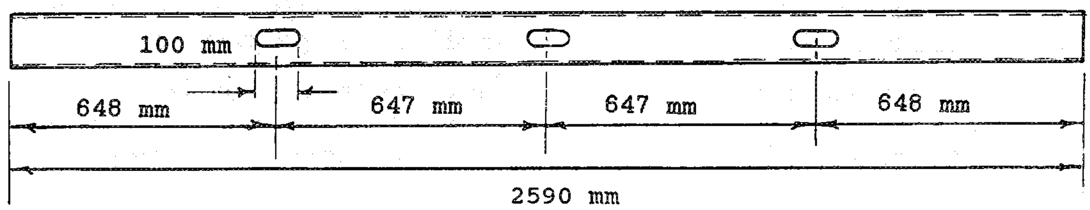


Figure 3.3A 20 Gauge

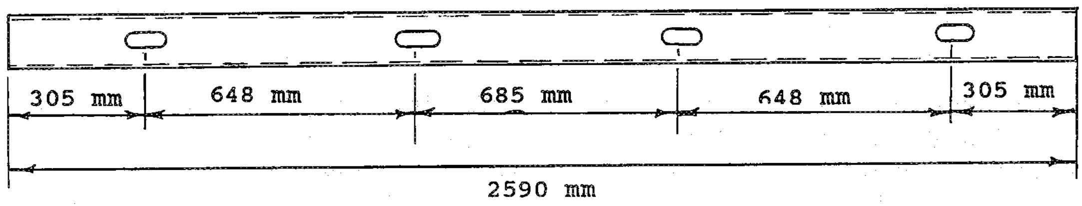


**Figure 18.8 Typical Types of Steel Stud Used in Test Program**

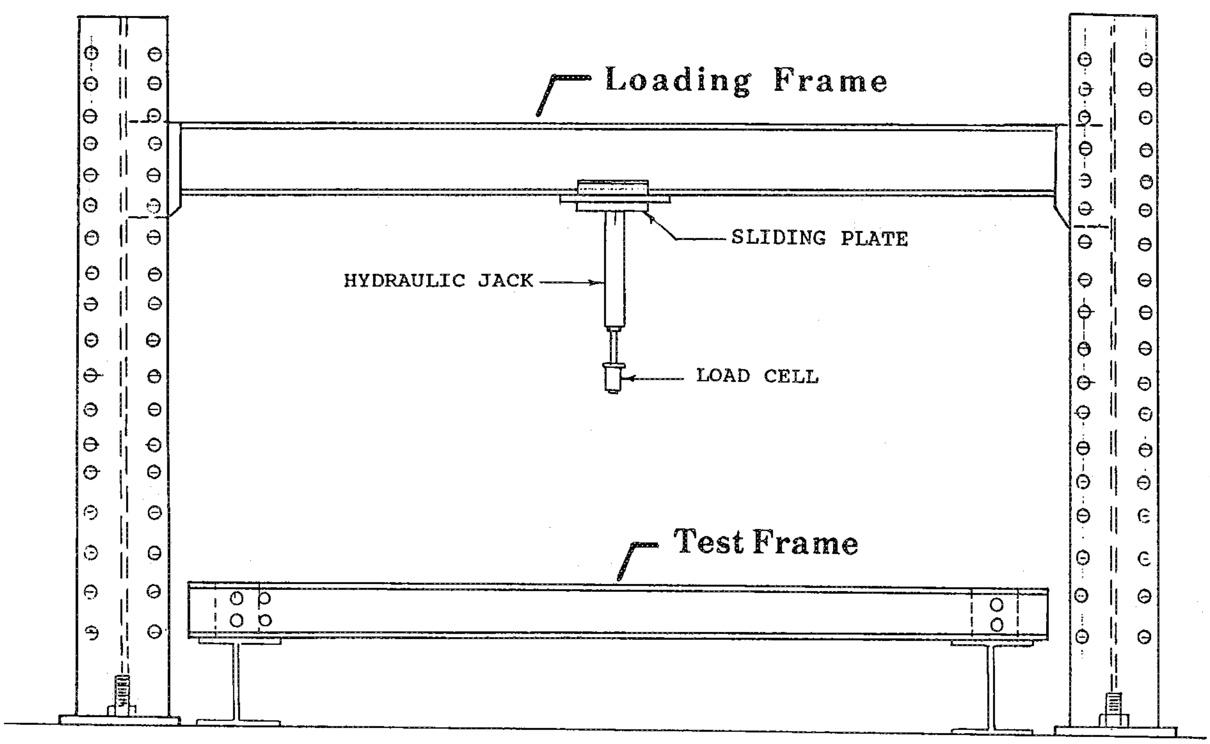


**Figure 18.9 Backup Wall Test Frame with Loading Frame in Position**

## TABLE 18.4 SUMMARY OF FAILURE LOADS PER STUD FOR WALL PANEL TESTS

| Specimen No. | Failure* Load $P_{tot}$ (N) | $P_{tot}/P_{avg}$ |
|---|---|---|
| 1-BW-1 | 1796 | 0.64 |
| 1-BW-2 | 1734 | 0.62 |
| 1-BW-3 | 1718 | 0.61 |
| 1-BW-4 | 1734 | 0.62 |
| 1-BW-5 | 1734 | 0.62 |
| 1-BW-6 | 1478 | 0.53 |
| 1-BW-7 | 2352 | 0.84 |
| 1-BW-8 | 2400 | 0.86 |
| 1-BW-9 | 2050 | 0.73 |
| 1-BW-10 | 1920 | 0.69 |
| 2-BW-1 | 3770 | 0.83 |
| 2-BW-2 | 3730 | 0.82 |
| 2-BW-3 | 3778 | 0.83 |
| 2-BW-4 | 4100 | 0.90 |
| 2-BW-5 | 3724 | 0.82 |
| 3-BW-1 | 3628 | 0.80 |
| 3-BW-2 | 3590 | 0.79 |
| 4-BW-1 | 2700 | 0.96 |
| 4-BW-2 | 3962 | 0.87 |
| 5-BW-1 | 1196 | 0.28 |
| 5-BW-2 | 1230 | 0.26 |
| 5-BW-3 | 1196 | 0.28 |
| 5-BW-4 | 1462 | 0.32 |
| 5-BW-5 | 1456 | 0.32 |
| 5-BW-6 | 1512 | 0.33 |

* Maximum value of each point load on stud equal two point loading on stud

- **Series No. 1**

In this series, ten 4 stud wide wall panels were fabricated with 20 gauge studs. The first five panels, specimens 1-BW-1 to 1-BW-5, were braced at midspan with 38.1 mm $\times$ 12.7 mm $\times$ 1.22 mm interior type steel bridging channels. As shown in Figure 18.10, the bridging was inserted through the midspan stud web cutout hole and fastened to the web of each stud with a 16 gauge clip angle and four number 8 self drilling screws. Specimen 1-BW-6 was identical to the first five except that only two screws were used in the connection. For specimens 1-BW-7 and 1-BW-8, two lines of interior steel bridging were used. Each line of bridging was inserted into the cutout holes at the quarter points and then attached to the studs in a manner similar to the first five tests specimens. The last two specimens (1-BW-9 and 1-BW-10) were fabricated identically to the first five specimens except that 12 mm thick interior gypsum board sheathing was also attached to the studs.

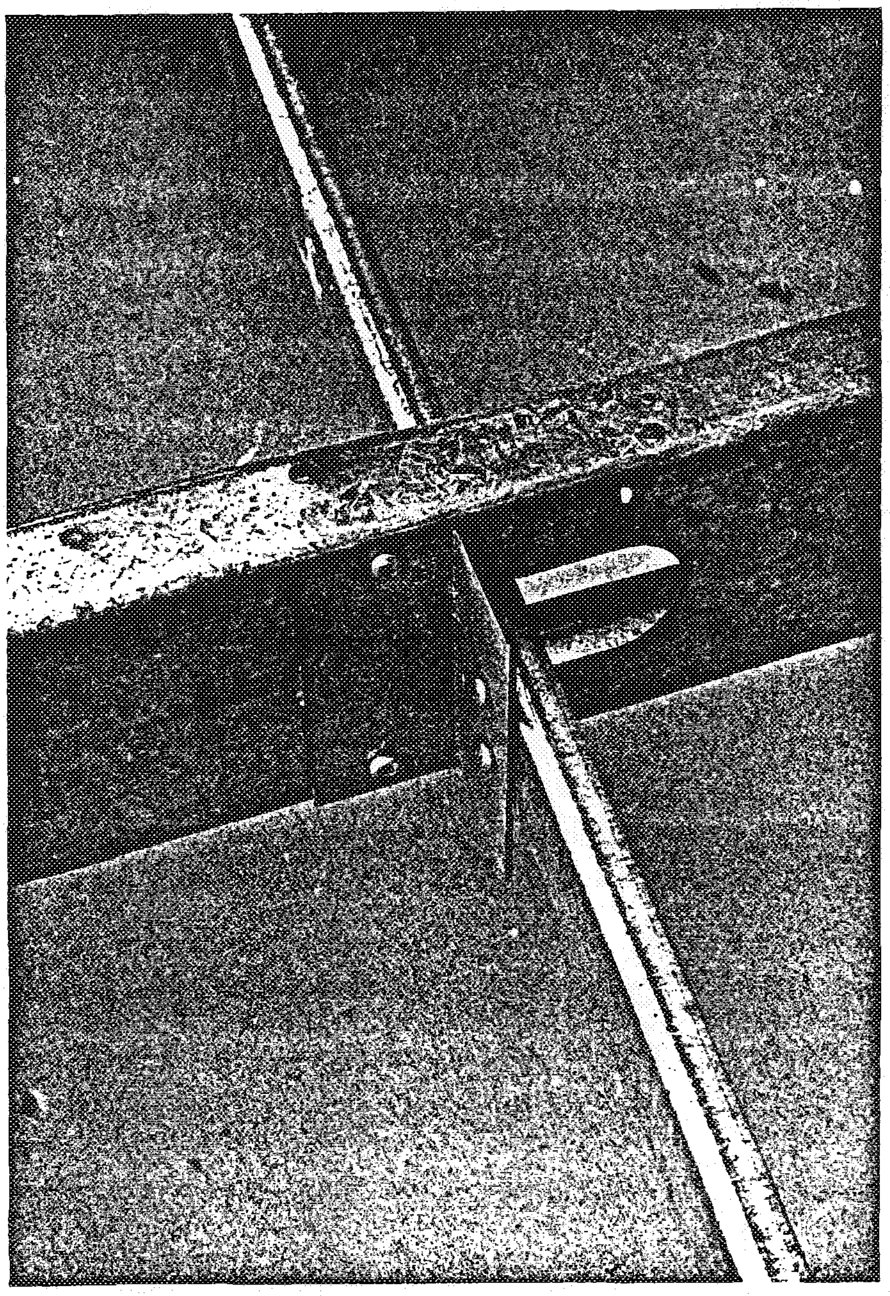

Figure 18.10 Photograph of Typical Clip Angle Bridging Connection Used in Test Program

For the first five tests, the midspan deflections of the interior studs are practically identical for each load increment plotted and are approximately linear up to load levels which caused approximately an L/180 midspan deflection. A tpical plot is provided in Figure 18.11.

For specimen 1-BW-7 failure occurred in the region of the midspan web cutout hole of both interior studs (Figure 18.12). Specimen 1-BW-8 failed in a similar manner.

For test specimens 1-BW-9 and 1-BW-10, simultaneous failure of both interior studs occurred in both tests. Each stud buckled in the region of the web cutout hole located midway between the midspan line of bridging and the bottom track. It was also noted that these studs exhibited significant local top flange buckling in the region of the midspan line of steel bridging.

- **Series No. 2**

Five 4 stud wide 18 gauge wall panels were fabricated in this series. Specimens 2-BW-1 to 2-BW-3 were braced at midspan with a line of exterior and interior 16 gauge face bridging. This type of bridging consisted of a 38.1 mm × 19 mm × 1.52 mm channel, knotched 406 mm on centre. The bridging was fastened to the exterior and interior stud flanges with a Number 8 Tek self drilling screw. The remaining two wall panels, 2-BW-4 and 2-BW-5, were braced at the quarter points with a line of face bridging attached to the exterior and interior flanges of the studs, in a manner similar to the first three specimens of this series.

The midspan deflections were linear and roughly identical for each load increment. The top and bottom stud end deflections were also approximately linear. In each of the first three tests the bottom track deflections.

Failure of specimens was signalled by the local buckling of one or both interior studs in the region of the web cutout hole located approximately 940 mm from the top track.

- **Series No. 3**

Series 3 consisted of two 4 stud wide 18 gauge wall panels braced at midspan with interior steel bridging. The bridging was attached to each stud with 16 gauge clip angles welded to each stud.

As in the other series the midspan deflection of each interior stud were nearly identical and linear for each load increment.

One interior stud collapsed in the region of the web cutout hole located between the centre line of bridging and the bottom track at failure of panel 3-BW-1. For 3-BW-2, buckle was in the region of the web cutout hole located between the centre line of the bridging and the top track on an interior stud.

- **Series No. 4**

In Series 4, two 4 stud wide wall panels were fabricated. Specimen 4-BW-1 was fabricated with 20 gauge steel studs bracked at midspan with steel bridging inserted through the midspan web cutout holes. In addition, the tension side of the panel was sheathed with 12 mm thick interior gypsum board fastened to the interior flanges of the studs with S-12 drywall screws, spaced 305 mm on centre.

For specimen 4-BW-2, it was observed that both interior studs were severely damaged in the region of the web cutout holes located 940 mm from the bottom track. One of the interior studs also exhibited some local deformation in the region of the web cutout hole located 940

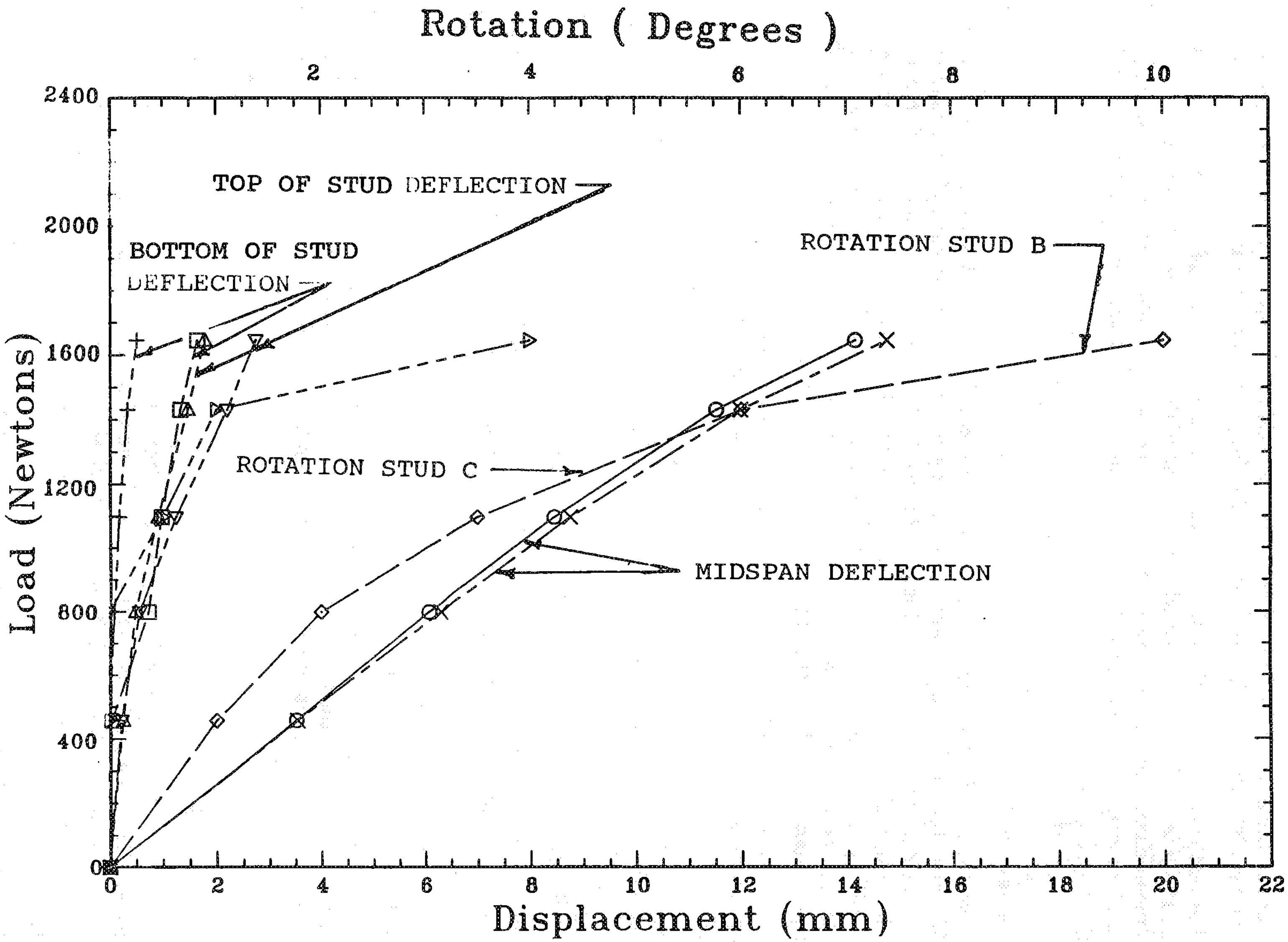


**Figure 18.11** **Load Versus Deflection and Rotation for Specimen 1-BW-1**

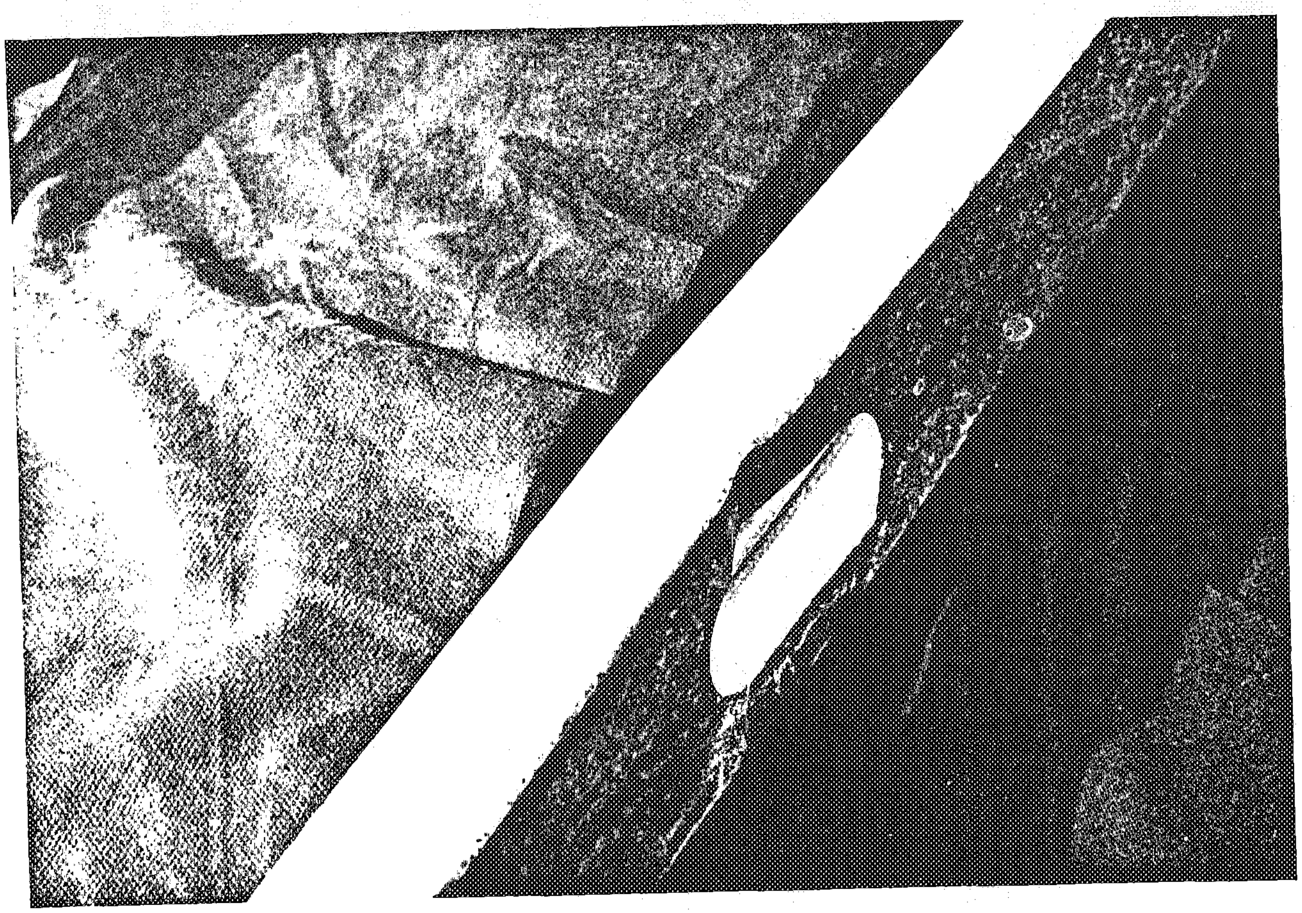

Figure 18.12 Photograph of Typical Local Buckling Failure of Interior Steel Stud - Series 1

mm from the top of the panel. One of the exterior studs also sustained damage in the region of the web cutout hole located 940 mm from the top of the panel.

- **Series No. 5**

The specimens tested in this series consisted of wall panels, two studs wide, unbraced between the top and bottom support tracks. Specimens 5-BW-1 to 5-BW-3 were fabricated with 20 guage studs and specimens 5-BW-4 to 5-BW-6 were fabricated with 18 gauge studs. Standard track was used at the top and bottom of each panel. Brick ties were fastened to the studs at the loading points.

Large midspan stud rotations occurred even at very small loads. The stud rotations were also non-linear and increased rapidly with load.

Specimens 5-BW-1 to 5-BW-3 all failed due to local buckling of the two steel studs in the region of the midspan cutout holes. For specimens 5-BW-4 to 5-BW-6, failure of each of these panels occurred when the two studs failed simultaneously in the region of of the web utout holes located approximately 940 mm from the top track support.

## 18.6 DISCUSSION OF RESULTS AND CONCLUSIONS

In general, as the number of lines of bridging increased there was a corresponding increase in the load carrying capacity of the steel studs.

In Series 5, steel bridging was not provided for the 20 and 18 gauge backup wall specimens tested. From the results shown in Table 18.4, for these tests it was found that the studs were only 26 to 33 percent efficient in carrying transverse loads and/or moment when compared to the beam capacities of 2795 N for 20 gauge studs and 4540 N for 18 gauge studs where torsional flexural buckling was presented.. In addition, large rotations occurred at the midspan of the steel studs.

For Series 1, 2, and 3, the 20 and 18 gauge specimens tested were braced with either one or two lines of steel bridging. This effectively increased the load carrying capacity to approximately 60 and 80 percent of the full capacity for the 20 and 18 gauge steel studs respectively, with one line of bridging at midspan. With two lines of bridging, one at each quarter point, the results showed an increase in load carrying capacity to approximately 84 and 90 percent of the full capacity for the 20 and 18 gauge steel studs respectively. The steel stud rotations decreased significantly with the addition of the steel bridging.

For a L/360 midspan deflection, the rotations which occurred were no larger than 2 to 3 degrees for the 20 gauge specimens and 1 to 2 degrees for the 18 gauge specimens. The commentary for the Canadian Code for cold formed steel structural members[7], suggests that steel bridging should be spaced at intervals no greater than that which will allow rotations in the order of 2 degrees. Rotations of magnitude greater than 2 degrees are thought to be objectionable in terms of serviceability requirements. For the backup wall panels tested with one line of bridging, the clear span between the midspan brace and either of the end supports was approximately 1280 mm. This would suggest that in order to limit rotations to a few degrees at service loads, the steel studs would have to be braced at intervals of 1280 mm or less.

In terms of bridging connections tested, it was found that the screwed clip angle connection shown in Figure 18.8 did increase the failure load to 60% or more of the flexural capacity of the stud if four screws were used to make the connection. It was observed that when the screws connecting the clip angle to the web of the stud were placed closer to the bend, less bending of the clip angle occurred. For these tests the 16 gauge clip angle used was found to be adequate. Preliminary tests with thinner clips showed that significant clip bending occurred.

However, there is a practical limit on how close the screws can be located to the bend in the clip angle. Based on the above facts it was concluded that the clip should be made of 16 gauge material or thicker and that the screws should be placed no further than one-third the distance of the leg, away from the bend. In order to control the screw location, pre-drilled holes should be provided in the leg of the clip angle which is connected to the web of the steel stud. The holes should be as far apart as possible since this would minimize the pullout force on the screws. Also, the clip angle should be approximately the same depth as the steel stud.

The welded connection detail was found to provide the stiffest connection as it allowed little to no rotation at the bridging location. This method should be considered for deeper and heavier steel studs since it is theoretically the strongest connection and would not be affected by improper installation or potential pullout of the screws.

The notched face bridging used in tests 2-BW-1 to 2-BW-5 was also found to increase failure load to a high percentage of the flexural capacity. However, care must be used to ensure that the ends of the lines of steel bridging are properly fastened to a building column or wall. This is necessary since it was found during a preliminary test that this type of bridging was not effective unless the ends of the bridging were adequately fastened. In general, the ends of any type of steel bridging should be adequately anchored.

The final item to be discussed deals with some visual observations made during the initial stages of the test program. When the steel studs arrived at the laboratory, it was oberved that a significant number of 20 gauge studs had sustained some visible damage. This was thought to have occurred due to the shipping and handling. An examination of the 18 gauge steel studs showed very little damage. During the fabrication process, it was also noticed that improper torquing of the sheet metal screws often led to stripped holes at the stud to track connection. This was more apparent in the 20 gauge material. Visits to some job sites in which steel studs were used inidicated that the 20 gauge steel studs sustained more damage on the site than the heavier gauges.

The above considerations as well as concern for tie connections and long term durability has contributed to the general conclusion that recommendations for good construction practice should specify 18 gauge minimum thickness.

## 18.7 RECOMMENDATIONS

Based on the results as presented in the complete report, the following recommendations are proposed:

1. The maximum spacing of track anchors should be 800 mm on centre.
2. For screwed stud to track connnections, the ends of the steel should be fastened to the support tracks with one self-drilling sheet metal screw in each flange.
3. Web crippling at the stud to track connection should be checked using the relevant Canadian design code provisions[6].
4. The track thickness should not be less than the thickness of steel stud.
5. While a minimum allowable thickness of steel stud and track of 0.91 mm (20 gauge) can be used, consideration of handling, storage and erection, and long term durability leads to the recommendation thay the minimum thickness be 1.22 mm (18 gauge).
6. The maximum spacing of steel bridging should be 1220 mm.
7. Generally, web cut-out holes should not be provided at locations other than where a line of steel bridging is to be provided. In particular, under no condition should there be a hole at the midspan of the steel stud unless a line of bridging is provided at this location. This recommendation applies only to lipped channel steel studs with web

cutout holes similar to that used in the test program. For other types of steel studs such as studs with regular openings in the web, it is recommended that load tests be performed under bending and torsional loading condition in order to establish the capacity of these types of studs. [Clause 9 of the code[6] should be consulted.] Where web cut-outs are required for services, they should be kept near the bottom of the wall where lower concentrated loads exist. As a rough empirical guide, these cut-outs should not be located in regions where the combined effects of bending and torsion under factored and load exceeds 60 percent of $F_y$.

8. Ties should not be located directly over web cut-out holes.

9. Clip angles used in steel bridging connections should be 16 gauge or thickner.

10. Screwed bridging to stud connections should be made with a minimum of four screws. The clip angle should be predrilled at the screw locations. The holes in the leg of the clip angle which is to be fastened to the web of the stud should be located no further than one-third the distance of the width of the clip angle away from the bend. These holes should be as far apart as possible since this will minimize the pullout force on the screws.

11. For heavier and deeper studs, a welded clip angle connection is suggested.

12. In terms of serviceability requirements, the maximum stud deflection should not be greater than L/720 under full design wind load. Other experimental research is ongoing to evaluate leakage rates through cracked brick veneer.

## REFERENCES


1. Arumala, J.O., Brown, R.H., "Performance Evaluation of Brick Veneer and Steel Stud Backup", Department of Civil Engineering, Clemson University, Clemson, North Carolina, 1982.

2. Baker, L.R., "The Flexural Action of Masonry Structures Under Lateral Load", Ph.D. Thesis, Deakin University, 1981.

3. Black, M.M., "Thin Walled Steel Structures", Gordon and Breach, Science Publishers Inc., New York, 1969, pp. 173-196.

4. Brick Institute of America, "Technical Notes on Brick Construction", No. 28B, Revision II, 1987.

5. Brook, M.S., "Masonry Veneer with Steel Stud Backup Wall", M. Eng. Project Report, McMaster University, April 1985.

6. "Cold Formed Steel Structural Members", CSA Standard CAN3-S136-M84, 1984.

7. "Commentary on CSA Standard CAN3-S136-M84", CSA Special Publication S136. 1-M86, 1986.

8. Dawe, D.J., "Matrix and Finite Element Displacement Analysis of Structures", Clarendon Press, Oxford, 1984.

9. Drysdale, R.G., "Seminar on BV/SS Research", McMaster University, November 9, 1988.

10. Drysdale, R.G., Gazzola, E., "Influence of Mortar Properties on the Tensile Bond Strength of Brick Masonry", Proceedings of the 7th Australian Conference, 1985.

11. Drysdale, R.G., Suter, G.T., "Modern Engineering Masonry".

12. Drysdale, R.G., Wilson, M.J., "A Report on the Behaviour of Masonry Veneer/Steel Stud Tie Systems", C.M.H.C. Report (in Press) 1989.

13. Galambos, T.V., "Structural Members and Frames", Prentice-Hall Inc., Englewood Cliffs, New Jersey, 1968.

14. Gjelsuik, A., "The Theory of Thin Walled Bars", John Wiley and Sons Inc., New York, 1981.

15. Grim, C.T., "Dimensional Stability of Clay Masonry", Internal Report, University of Texas, Austin, 1981.

16. Hatzinikolas, M., Lee, R., Longworth J., and Warwaruk, J., "The Behaviour of Styrofoam SM Insulation as Exterior Sheating in Brick Veneer and Metal Stud Wall Systems", Dept. of Civil Engineering, University of Civil Engineering,University of Alberta, 1986.

17. Hetnakul, N., Yu, W.W., "Webs for Cold-Formed Steel Flexural Members and Structural Behaviour of Beam Webs Subjected to Web Crippling and Combination of Web Crippling and Bending", University of Missouri-Rolla, June 1978.

18. Hill, H.N., "Lateral Buckling of Channels and Z Beams", ASCE Transactions Paper No. 2700, 1954, pp. 826-841.

19. Lansing, W., "Stresses in Thin Walled Open Section Beams Due to Combined Torsion and Flexure", Ph.D. Thesis, Cornell University, June 1949.

20. Lawrence, S.J., "Behaviour of Brick Masonry Panels Under Lateral Loading", Ph.D. Thesis, University of New South Wales, November 1983.

21. "Lightweight Steel Framing Manual", Canadian Sheet Steel Building Institute Publication, Willowdale, Ontario, August 1987.

22. "Masonry Design for Buildings", CSA Standard CAN3-S304-M-84.

23. Metal Lath/Steel Framing Association, "Here are the Facts", Publication, Chicago, Illinois.

24. Murden, J.A., "Evaluation of Lateral Stiffness for Gypsum-Faced Steel Stud Backup Panels", Master of Science Thesis, Department of Civil Engineering, Clemson University, Clemson, North Carolina, August, 1984.

25. "Steel Framing Systems", Baily Metal Products Ltd., Publication, Toronto Ontario.

26. Tong, P., Rossetos, J.N., "Finite Element Method", MIT Press, Cambridge, Massachusetts, 1977.

27. "Torsional Analysis of Steel Members", American Institute of Steel Construction, Chicago, Ill., 1983.

28. "USG Steel Framing Systems", United States Gypsum Company Publication, 1986.

29. Vlasov, U.A., "Thin Walled Elastic Beams", Israel Program for Scientific Translations, Jerusalem, 1961.

30. "Wheeling Steel Framing", Wheeling Corrugating Company, Wheeling, West Virginia, Catalogue 1981 Edition.

31. Winter, G., Lansing W., and McCalley, R.B., "Performance of Laterally Loaded Channel Beams", Symposium on Engineering Structures (Colston Papers, Vol. 2) Butterworths Scientific Publications, London, 1949.

32. Winter, G., Pian, R.H., "Crushing Strength of Thin Steel Webs", Engineering Experiment Station Bulletin No. 35, Part 1, Cornell University, Ithaca, New York, April 1946.

33. Yu, W.W., "Cold Formed Steel Design", John Wiley and Sons Inc., New York, 1985.

34. Zbirohowjki, K., "Thin Walled Beams", Crosby Lockwood and Son Ltd., London, 1967.

35. Zetlin, L., "Elastic Instability of Flat Plates Subjected to Partial Edge Loads", Journal of the Structural Division, ASCE Proceedings, Vol. 81, September 1955.

# CHAPTER 19

# A SUMMARY OF

# WATER LEAKAGE CHARACTERISTICS OF BRICKS AND BRICK ASSEMBLAGES

by

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and

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Graduate Student

## 19.1 INTRODUCTION

This report contains a summary of a laboratory investigation into the permeability of masonry veneers.

The ability of masonry walls to resist water penetration has been well documented since the 1930's by researchers such as Palmer & Parsons[4], McBurney et al[3], Connor[1], Fishburn[2] and Ritchie[5]. Although much of their work is still applicable today, the relatively new veneer wall system incorporating a flexible steel stud back-up has introduced another factor in the water penetration potential of masonry veneers and that is the possibility of flexural veneer cracking under service loads. If researchers and building owners have found that uncracked masonry can leak, how much more potential for leakage is there for a cracked veneer? This question provides the context for the water permeability testing of brick/brick assemblages as summarized in this chapter.

## 19.2 TEST METHOD

The test method developed to evaluate brick/brick assemblage leakage characteristics is an adaptation of that of Palmer & Parsons[4] in which the results were obtained in a quantitative form and in which brick assemblages were tested in an uncracked and then cracked state.

The objectives set forward to develop a standard quantitative test method were as follows:

- Test method to cover both brick unit and assemblages
- Assemblage must have a representative ratio of mortar joint area to unit area
- Water pressure head on specimen of 25 and 100 mm
- Constant head during test
- No lateral water flow at specimen perimeter
- Separation of flow through assemblage from flow escaping at specimen perimeter
- Collection of leakage water for weighing
- Minimize evaporation of leakage water
- Crack assemblage along a single mortar joint
- Adjust crack to a specified shape and dimension

Much effort and many hours were consumed in developing a test method to satisfy these objectives. The greatest problems encountered were eliminating perimeter leakage and adjusting the crack to a predetermined size and shape.

The method developed satisfies all of the objectives although it was found that some compromise was necessary with regards to the degree of control of crack size and shape for very small crack widths.

In terms of the brick assemblage specimens, the method involves preparing specimens with representative ratios of mortar joint area to brick area (Figure 19.1) so that the exterior brick face can be held horizontally with a head of water retained on top (Figure 19.2). Both 25 mm and 100 mm heads of water representing water pressures of 245 Pa (5.1 psf) and 980 Pa (20.5 psf) respectively were investigated for uncracked specimens. Cracked specimens were subjected to a 25 mm head of water only. Cracking of the specimen was accomplished such that the crack occurred along the middle mortar bed joint. Turnbuckles were then used to adjust the crack size and shape . Measurement using mechanical gauge points on either side of the crack were used to control the crack size and shape.

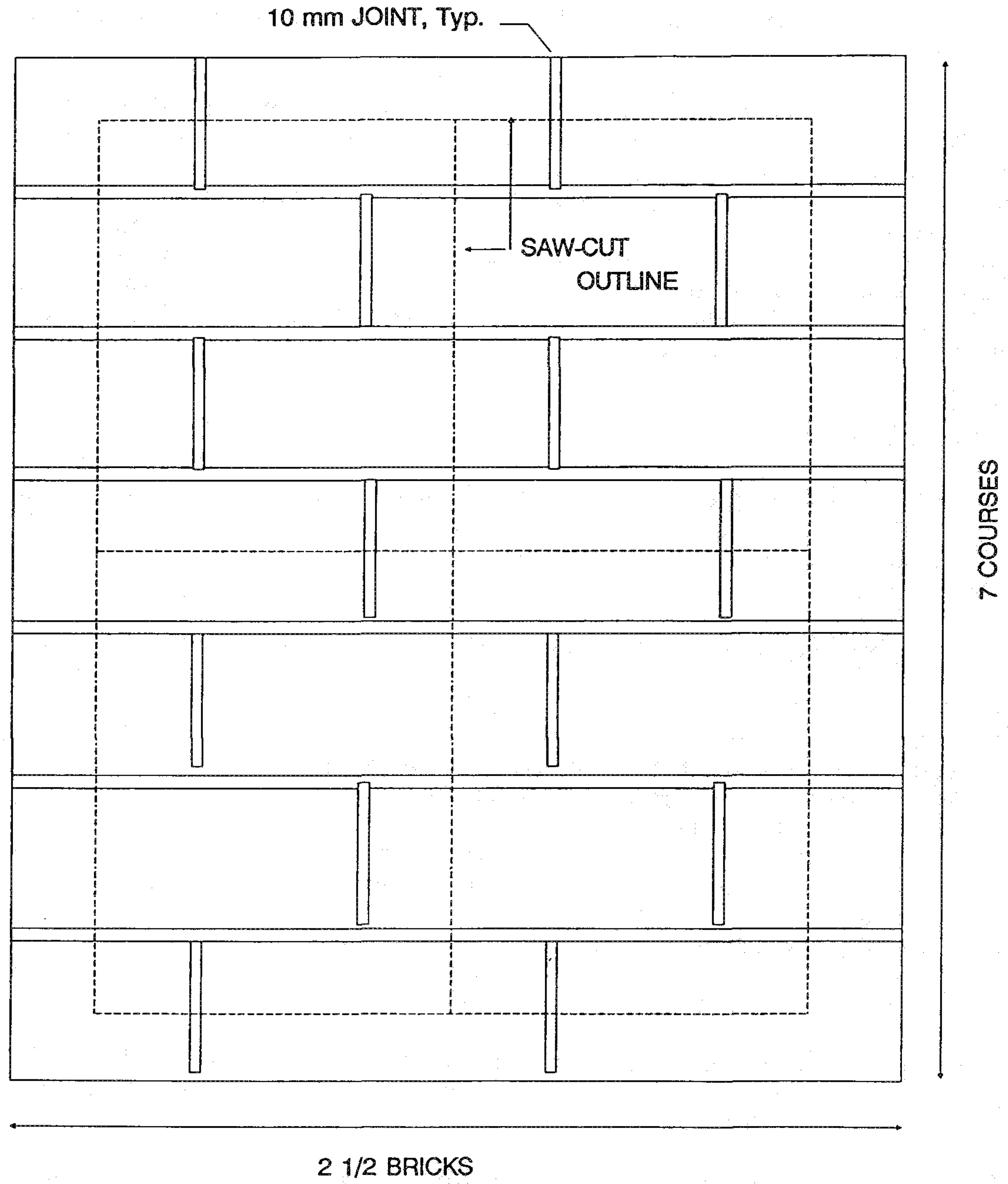


FIGURE 19.1 LEAKAGE SPECIMENS CUT FROM BRICK WALLETTE

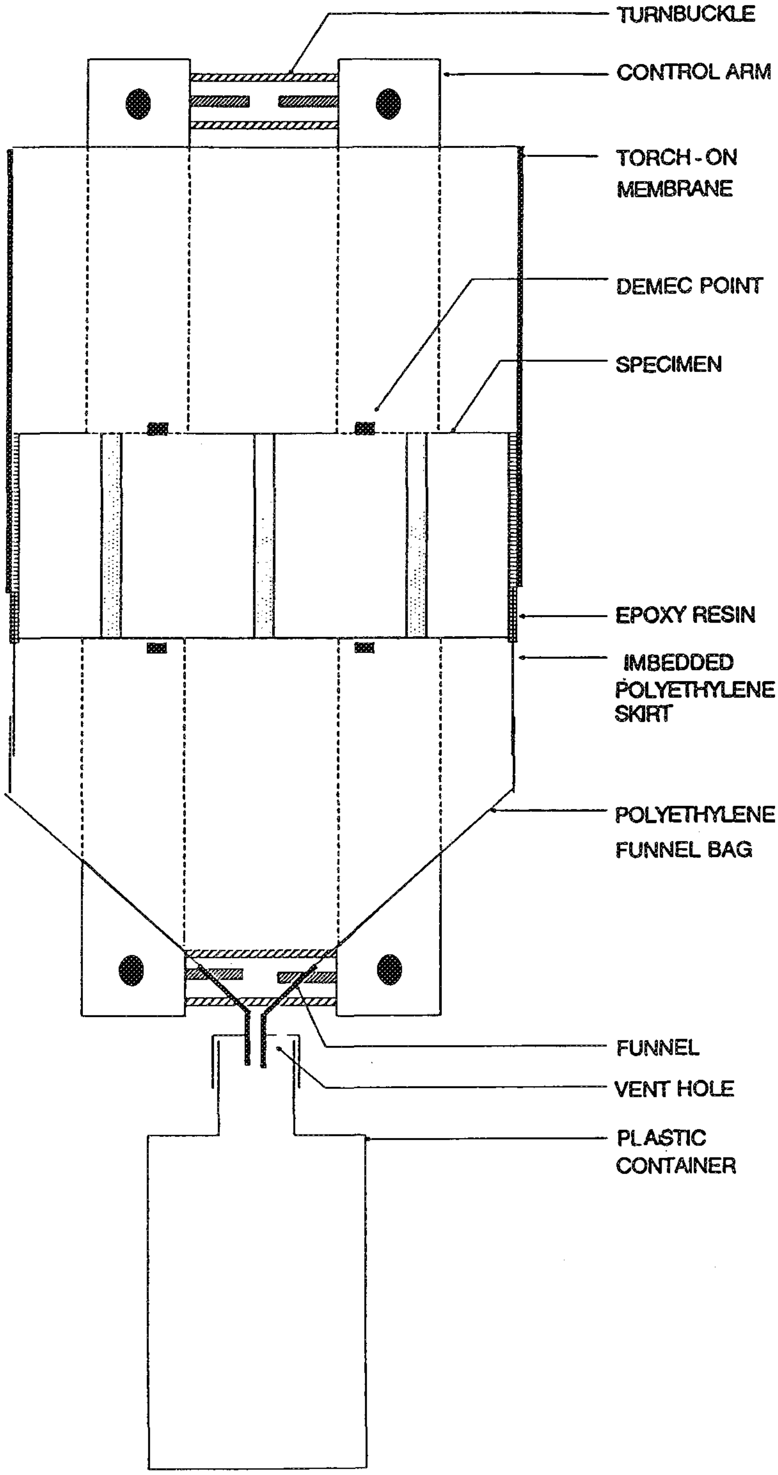


FIGURE 19.2 BRICK ASSEMBLAGE TEST SET-UP

## 19.3 TEST SPECIMENS AND RESULTS

### 19.3.1 Brick Units

Two types of clay and four types of concrete brick units were investigated. All were of high quality with no fire cracking evident in the clay bricks and water retardents in the concrete bricks. All were found to be essentially water tight.

### 19.4.2 Brick Assemblages

Brick assemblages were prepared from the 6 types of bricks previously mentioned with combinations including 3 types of mortar and full and partly filled joints.

Typically it was found that uncracked clay assemblages reached a steady state of leakage as seen in Figure 19.3 for three specimens. After cracking, leakage peaked but then dropped off very quickly. Uncracked concrete assembalges tended to peak at the beginning of the test and then drop off (Figure 19.4). This was repeated after cracking.

In both the cracked clay and uncracked and cracked concrete cases, the phenomena of a peak flow followed by a drop in flow indicates a certain amount of self healing occurring in the specimen.

## 19.4 CLOSURE

It must be stressed that these are general observations made from data that is extremely scattered.

## REFERENCES


1. Connor,C.C., "Factors in the Resistance of Brick Masonry Walls to Moisture Penetration", ASTM Proceedings, Vol 48, June 1948.

2. Fishburn,C.C., "Water Permeability of Walls Built of Masonry Units", NBS Building Materials and Structures Report 82, April 1942.

3. McBurney,J.W., Copland,M.A., Brink,R.C., "Permeability of Brick-Mortar Assemblages", ASTM June 1946.

4. Palmer,L.A., Parsons,D.A., "Permeability of 8-in Brick Wallettes", ASTM Proceedings, Vol. 34, part II, 1934.

5. Ritchie,T., Plewes,W.G., "Moisture Penetration of Brick Masonry Panels", ASTM Proceedings, Oct.1960.

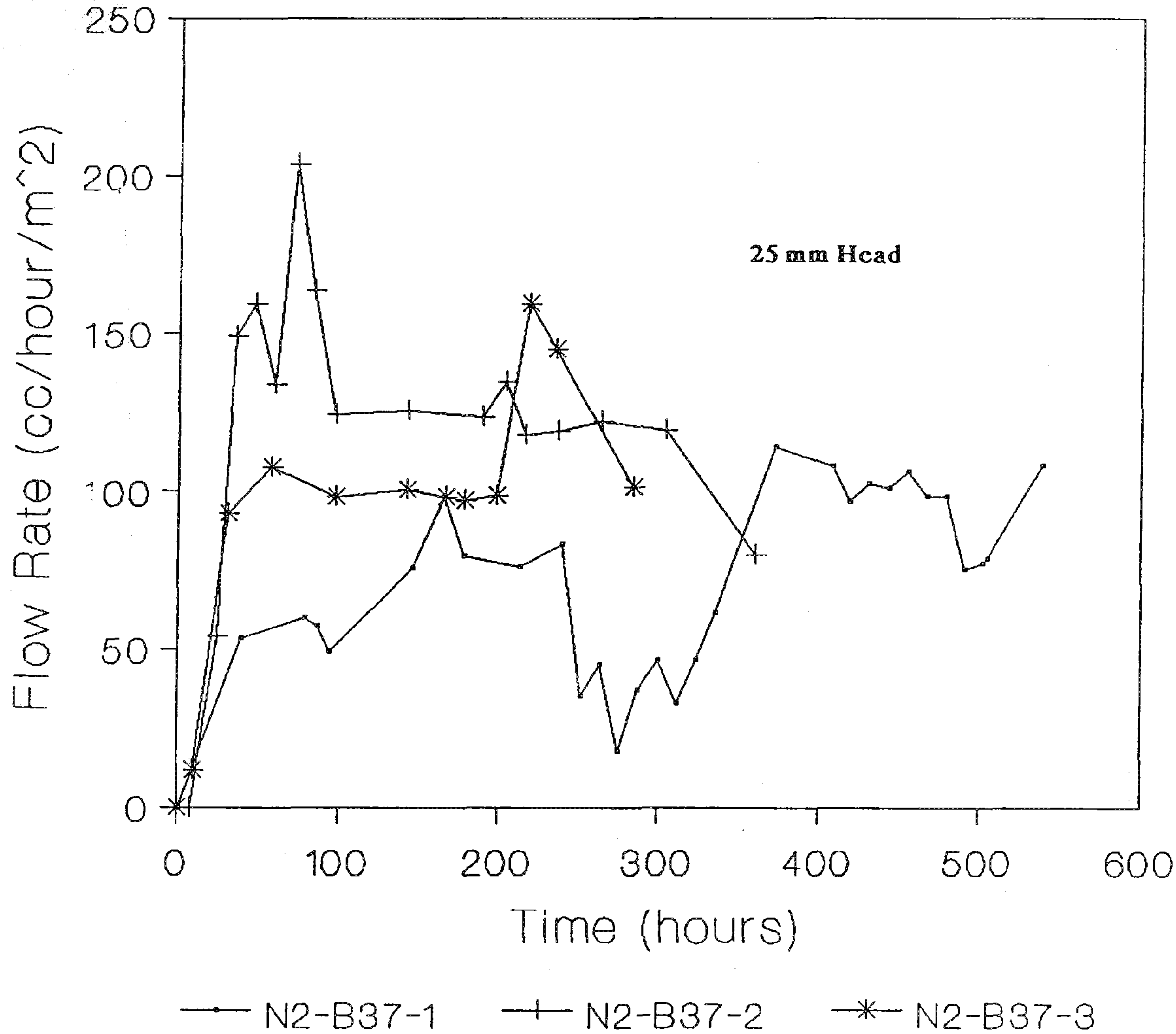


**FIGURE 19.3 AVERAGE FLOW vs TIME FOR 3 CLAY BRICK SPECIMENS - N2-B37 1-3**

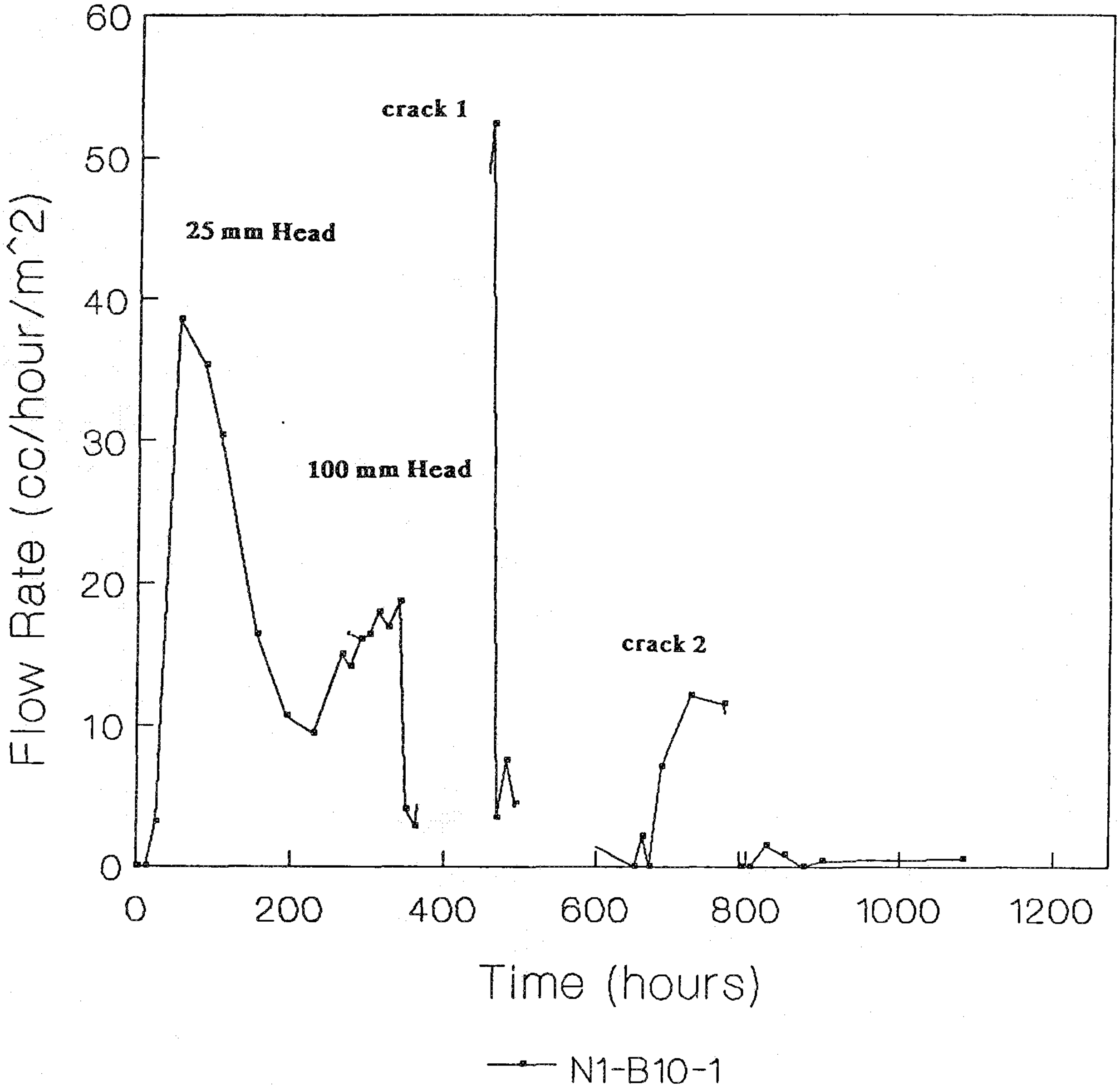


**FIGURE 19.4 AVERAGE FLOW vs TIME FOR ONE CONCRETE SPECIMEN - N1-B10 1**

CHAPTER 20

# A SUMMARY OF

# PERFORMANCE OF BRICK VENEER/STEEL STUD WALL SYSTEMS SUBJECT TO TEMPERATURE, AIR PRESSURE AND VAPOUR PRESSURE DIFFERENTIALS

by

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Professor

and

Andrew Kluge
Graduate Student

## 20.1 INTRODUCTION

This chapter is a summary of research information on the environmental performance of brick veneer / steel stud (BV/SS) wall systems. The research comprises Part 3 of a five part laboratory test program conducted at McMaster University and sponsored by the Project Implementation Division of Canada Mortgage and Housing Corporation. Although the research focused on BV/SS wall systems, the apparatus developed to conduct the experiments ( Environmental Simulation Test Apparatus ) and the results obtained have significance for wall systems in general.

The three key players in wall system performance from an interior humidity perspective are air pressure, thermal and vapour pressure differences. The objective of this test program was to investigate the performance of BV/SS wall systems subjected to the individual factors of air pressure and thermal gradients and then to the combined effect of all three.

Before any of this testing could be done, it was necessary to design and build an apparatus capable of performing these tests in an economical fashion. The apparatus built, termed the Environmental Simulation Test Apparatus[12] (ESTA) is capable of testing $1m^2$ wall specimens under any or all of the three conditions as well as simulated rain. This unique piece of equipment allows many combinations of wall systems to be tested for a very reasonable price due to the small test area.

With ESTA it was then possible to investigate points of interest such as; the amount of air flow that can occur through a range of opening sizes in the air barrier; thermal profiles, especially in the vicinity of bridging components; and accumulation of condensation and any resulting damage.

Within the scope of the test program were standard BV/SS wall systems with and without an insulating exterior sheathing as well as a BV/SS wall having a non-standard framing system. Conditions of testing were kept to values which are reasonably expected. In tests for moisture accumulation due to condensation, either intentional or unintentional leakage paths were included in acknowledgement of the fact that it is physically impossible to build a building with a perfect air barrier.

## 20.2 THE ENVIRONMENTAL SIMULATION TEST APPARATUS

Over the course of the investigation summarized in this chapter, two versions of ESTA, ESTA 1 and ESTA 2, were constructed and used. Both versions are briefly described in this section.

### 20.2.1 ESTA 1

ESTA 1[12] consisted of a Load Box, Specimen Box and Cold Box. Figure 20.1 is a schematic drawing of the basic components of ESTA 1 as arranged for moisture accumulation testing.

In this particular configuration, the Load Box is used to simulate an interior environment while in the Cold Box, exterior winter conditions are simulated.

In the case of rain penetration testing, the Load Box is attached to the other side of the Specimen Box and is used to contain the water spray gear and the run off water. This versatility allows the same wall specimen to be tested for all three types of tests without disturbing its location in the specimen box.

Measurement and control systems included air pressure, air flow rate, temperature and humidity in the load box and temperature in the cold box. Data acquisition was achieved by manual means for pressure, humidity and air flow and by a computer automated method for the wall temperature profile. The main purpose of this research, that of moisture accumulation testing, was economically accomplished with ESTA 1. With the addition to the setup of a mechanical cooling system, ESTA 1 wasbrought into the realm of a low cost yet versatile and productive piece of test equipment.

After experience was gained in using the equipment, two problems were realized. The first was the inability to reduce the incidental apparatus leakage to one order of magnitude below the wall leakage for relatively air tight wall specimens. It was difficult to pin point where this apparatus leakage was occurring.

The second problem was associated with attachment and detachment of the Load Box from the Specimen Box. The process was lengthy and there was some question as to the repeatability attainable in one detach-attach cycle.

For these reasons a second version of ESTA was built and is described in the next section.

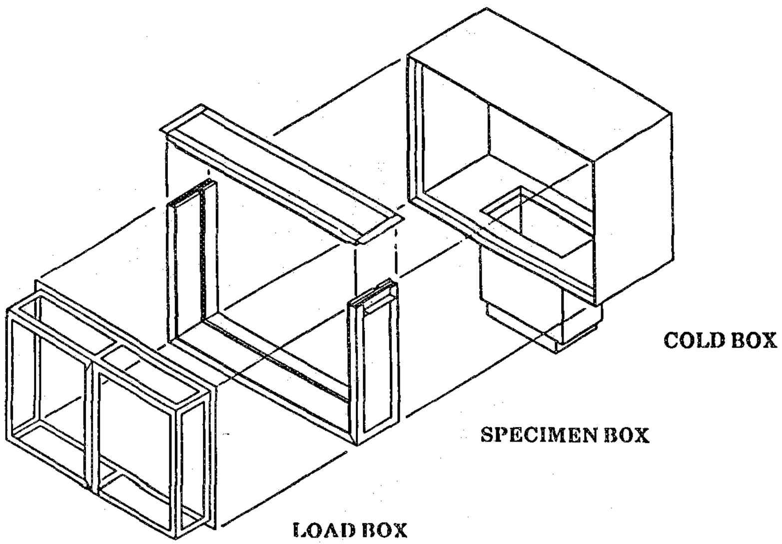


**FIGURE 20.1 ENVIRONMENTAL SIMULATION TEST APPARATUS**

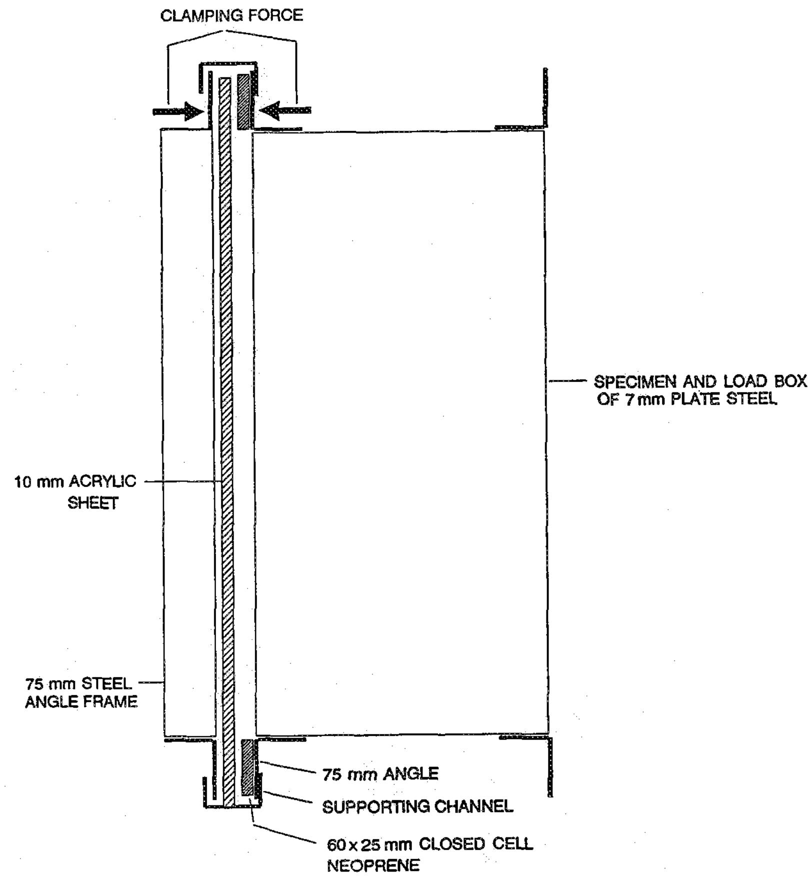


**FIGURE 20.2**

**VERTICAL SECTION THROUGH ESTA 2**

### 20.2.2 ESTA 2

ESTA 2 was built with the same design parameters as ESTA 1 yet in a simpler fashion. The Load Box and Specimen Box are one unit and consist of a 7 mm thick steel plate box lined with 75 mm of polystyrene insulation. Seventy-five millimeter steel angles welded to the outer perimeter of the Specimen Box on both sides serve as reaction points for clamping devices as shown Figure 20.2. A 75 mm angle frame matching the dimensions of the stud angle framing on the Specimen Box is used to rigidly support a 9 mm acrylic sheet which covers the open area of the Specimen Box. Sealing at the perimeter is accomplished with a 60 x 25 mm closed cell neoprene material as depicted in Figure 20.2. Clamps are used as quick connect and disconnect features to compress the neoprene seal. The mechanically cooled Cold Box remained unchanged.

ESTA 2 is a much improved version of the apparatus for both air tightness and ease of use. The acrylic sheet slides in and out for easy access. A maximum of 6 clamping devices are needed to achieve an essentially air tight fit. Repeatabilty between opening and closing the Specimen Box was found to be excellent.

## 20.3 TEST PROCEDURES

### 20.3.1 General

Since temperature distributions and air leakage characteristics are individual factors which play key roles in moisture accumulation, tests were conducted to document these properties before moisture accumulation tests were performed. The main features of each type of test are briefly described in the following sections.

### 20.3.2 Air Leakage Test

Air leakage tests were performed to determine the air flow rate through wall systems under a range of pressure differentials. Since air barriers are required by the 1985 version of the NBC, this type of testing in the context of modern wall systems is primarily aimed at quantifying the air tightness of the air barrier, although other components of the wall system can have some effect on restricting air flow.

The two approaches taken in this work were:

1. Introduce a well defined leakage path in a 'perfect' air barrier.
2. Construct the wall system according to specifications and quantitatively determine the effectiveness of the air barrier.

In either case the method involved applying a known pressure differential across the wall system and measuring the air flow rate, or, conversely, setting the air flow rate and measuring the pressure differential.

### 20.3.3 Thermal Performance

Two types of thermal profiles were measured. The predominate type was through-the-wall at each material interface. A typical through-the-wall profile consisted of measurements at the following locations:

-interior air
-air/gypsum
-gypsum/insulation or stud
-insulation or stud/exterior sheathing
-exterior sheathing/air cavity
-cavity air
-air cavity/brick
-brick/air
-exterior air

Through-the-wall profiles were normally taken to coincide with the line of the stud or at the lateral centre line of the insulation mid way between studs. A second type of profile was taken in-the-plane of the wall at the interface between the stud or stud space insulation and the interior side of the exterior sheathing. A third type of thermal measurements performed involved taking local measurement at points of interest.

Since every measured profile had slightly different interior and exterior temperature conditions, a method similar to that of Sasaki[18] was used to facilitate comparison between results. This involved calculating a non-dimensional coefficient for each thermocouple location as:

$$X_X = 1 - \frac{(Tint - T_X)}{Tint - Text} \qquad (20.1)$$

where $X_X$ = non-dimensional coefficient for location X
Tint = interior temperature
Text = exterior temperature
$T_X$ = temperature at point X

In this way, each measured point has a value ranging from 1 (interior) to 0 (exterior). Thus, by rearranging the equation, the temperature at any point becomes,

$$T_X = (Tint-Text)(X_X-1) + Tint \qquad (20.2)$$

In the report, the thermal results are presented in this fashion for comparative purposes along with the average $X_X$ values used to obtain the profiles.

### 20.3.4 Moisture Accumulation Testing

The main purpose of this test was to examine the potential for condensation and to assess vulnerability of various wall systems to condensation and condensation related damage. The testing involved:

- creating a pressure differential across the wall and measuring the resulting air flow rate or alternately setting a desired flow rate and measuring differential pressures,
- maintaining a specified and constant level of humidity on the interior of the wall system
- providing a constant temperature differential across the wall and measuring temperatures at set locations and
- examining the wall during and after the test for evidence of condensation.

The details of exactly how these conditions were accomplished for each test are explained as the test results are presented.

## 20.4 WALL TEST SPECIMENS

Three types of brick veneer/steel stud wall systems were investigated. Listing the components from the interior to the exterior, each wall type consisted of interior gypsum board, a polyethylene vapour barrier, a steel stud framing system with insulation in the space between studs, a sheathing material on the exterior of the studs, an air space between sheathing material and brick veneer (cavity), and a brick veneer tied by brick ties to the steel stud framing system.

The basic differences between the three wall types investigated can be summarized as follows:

- Wall Type 1 (Figure 20.3) was built with an exterior grade gypsum board sheathing material which by its nature has essentially no insulating value.
- In contrast, Wall Type 2 was built with an insulating polystyrene sheathing material.
- Wall Type 3 was different from Type 1 and 2 in that the steel framing system was non-standard (Figure 20.4). Furthermore, a range of insulation schemes was investigated to obtain an optimum configuration.

In summary, the three types of wall systems investigated in this work could be classified as having a non-insulating sheathing, an insulating sheathing and a non-standard steel framing system. Two Type 1 and 2 walls (Specimen 1A, 1B, 2A and 2B) were tested while one Type 3 wall (Specimen 3) was tested.

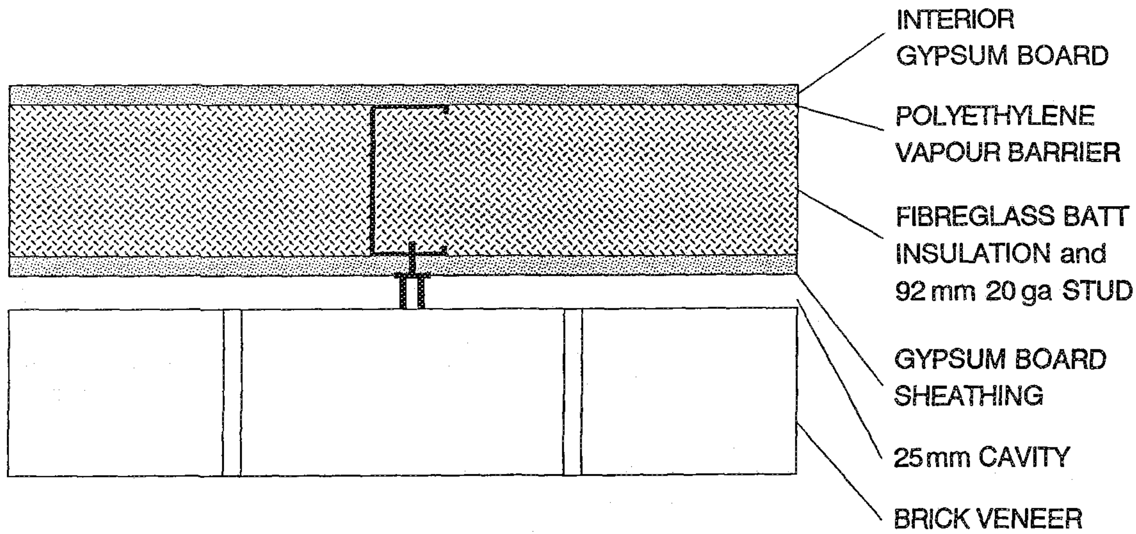


**FIGURE 20.3 HORIZONTAL SECTION SHOWING COMPONENTS WALL TYPE 1**

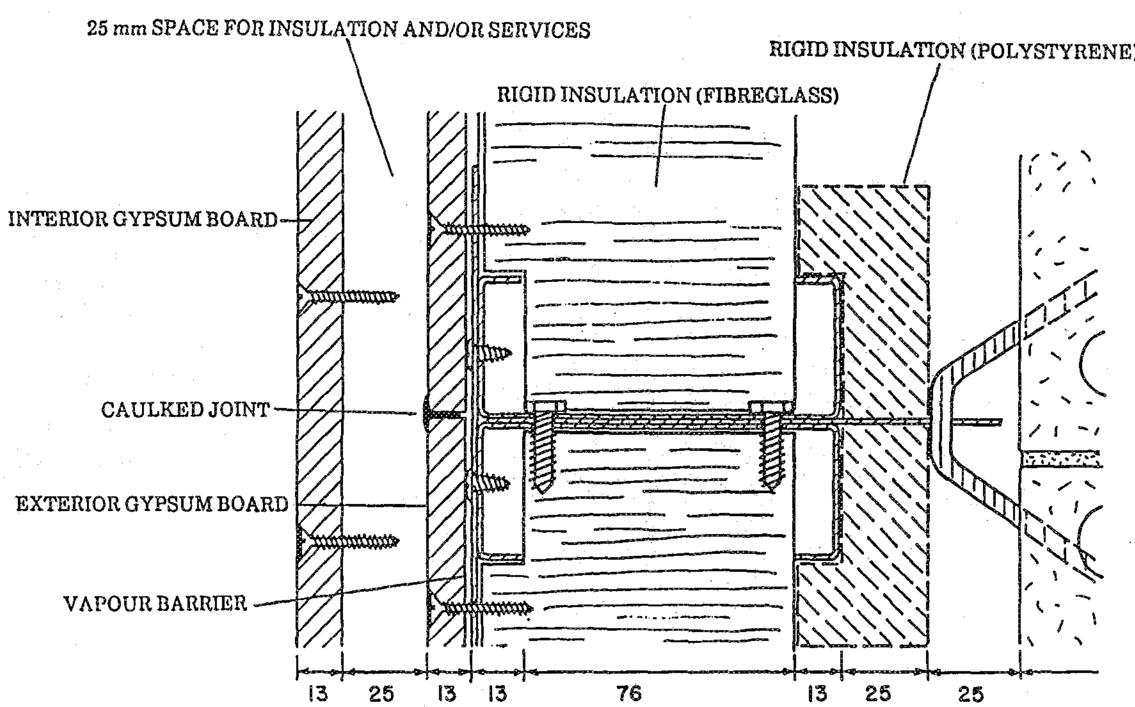


**FIGURE 20.4 HORIZONTAL SECTION SHOWING COMPONENTS WALL TYPE 3**

### 20.5 TEST RESULTS

Air leakage, thermal performance and moisture accumulation tests were performed on the specimens.In this brief report, only the the results of the moisture accumulation tests willbe summarized.

#### 20.5.1 Wall Specimen 1A

In the moisture accumulation test, an air pressure difference, a vapour pressure difference and a temperature gradient were maintained across the wall to simulate winter conditions. A net outward pressure of 75 Pa was chosen as the air pressure difference since such a pressure can occur from the action of stack effect alone in the upper floors of high rise buildings. A relative humidity of 35 -40%, considered representative for dwellings, was maintained.

With the 75 Pa pressure difference, air flowed through the wall at a rate of 0.015 L/sm$^2$.

Maintaining the temperature gradient proved to be a problem. At this stage the cooling was provided by an open top freezer in conjunction with dry ice. It was difficult to maintain the temperature at a steady condition. Thus, over the 1 week test duration, the cold side varied between a minimum of -17$^0$C and a maximum of -0$^0$C, while the warm side was kept at 21 $\pm$ 2$^0$C

After 7 days, the cooling system could not maintain a temperature below 0$^0$ and the wall was dismantled. Upon dismantling there were no signs of moisture on the stud, fasteners or in the batt insulation. There was, however, evidence that moisture had formed at some point in the test as corrosion was present in three distinct locations as detailed in Figure 20.5.

Figure 20.5 is a sectional view showing a fastener used to attach the exterior the 12.5 mm gypsum board to the steel studs. Corrosion was evident along the length of the portion of the fastener which protruded into the stud channel, on cuttings at the fastener tip and also on the head of the fastener exposed to the cavity. Corrosion was also present on the steel burs formed during installation of the screw fastener.

#### 20.5.2 Wall Specimen 1B

The testing for moisture accumulation in Wall Specimen 1B was performed over a period of 11 days. The conditions under which the test was run were as follows:

- air flow through the wall at a rate of 0.15 L/s.m$^2$
- temperature gradient of +21 to -20$^0$C
- interior humidity level of 40% RH

The intended air barrier in this specimen was the eyxterior gypsum board. To this end, the perimeter and all joints were sealed with silicone and/or tape. The exterior gypsum board was constructed with three panels spanning from Stud A to Stud B as well as two sides panels.

Since the exterior gypsum board was the intended air barrier, a rather large leakage path on the vertical perimeter of the interior gypsum board closest to Stud A was permitted by not applying sealant. The resulting opening was about 800 mm long (full height) and 2 mm wide. After running the test for 11 days under the above conditions, weight gain in the gypsum panels was recorded as 6, 16, 41 and 109 grams for panel number 2 to 5 respectively. This gain in weight was a result of moisture gain from condensation.

There was no noticable weight gain in the batt insulation. However, as was the case with Wall Specimen 1A, corrosion was present on the fasteners attaching the exterior gypsum board to the steel stud as well as on the burs around the fastener hole in the stud.

#### 20.5.2 Wall Specimen 2A

Moisture accumulation testing on Specimen 2A was performed in four parts. Only Part 4 will be reported on here. At this point in the test program the chest freezer/dry ice cooling combination was replaced with a new mechanical cooling system. This new system was capable of maintaining -20$^0$C and incorporated programmable evaporator coil defrost cycles.

Part 4 was performed with this new cooling system. For this test the main concern was to investigate the effect of workmanship in the installation of the rigid insulation by simulating a 3 mm

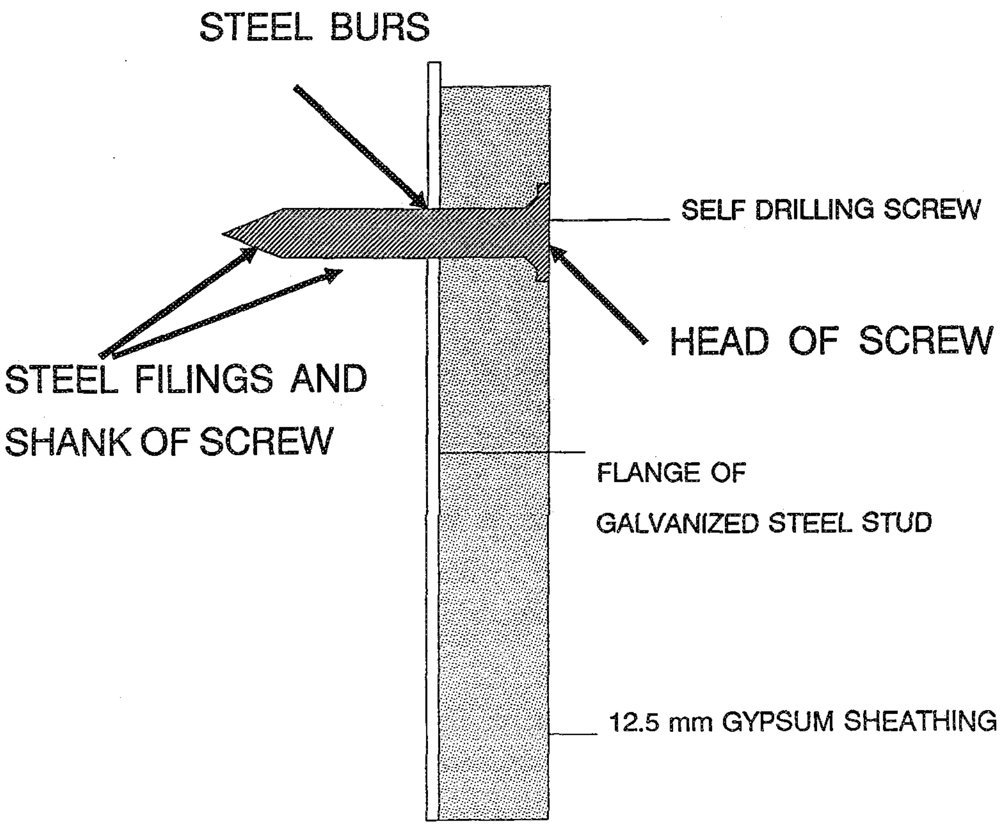


**FIGURE 20.5 LOCATION OF CORROSION PRODUCTS**

gap at a joint between insulaton boards. This was accomplished by simply cutting a 3 mm wide slot, extending horizontally from Stud A to Stud B at the joint between boards.

The test was run with an interior RH of 50-55% and a leakage path flow through the wall of 0.007 L/sm$^2$. The exterior temperature was maintained continuously at about -21$^0$C save for 20 minute defrost cycles every 6 hours. After 4 days the wall was dismantled. At this time the batt insulation was frozen to the polystyrene board in the same area that was found moist in Part 3 (Figure 20.6). After the batt was allowed to warm up a little, it was noted that moisture could be felt on the exterior batt face to a depth of approximately 12 mm. The slot cut into the polystyrene wa frosted shut (Figure 20.7) along with the two upper vent holes in the brick veneer (Figure 20.8). The two lower weep holes in the veneer had no signs of frost.

There was no sign of moisture accumulation on the studs. However, upon melting of the frost, water flowed down the interior face of the polystyrene board to the bottom track. In this test, the track lip was not in contact with the polystyrene (manufactured with an inward bend) and the moisture ran between the track and the board.

### 20.5.4 Wall Specimen 3

Having evaluated the five different insulation configurations listed in Table 20.1 in terms of thermal performance the optimum configuration was chosen as configuration E. With this wall system, the environmental performance was examined in terms of the potential for moisture accumulation. The test was run for 120 hrs (5 days).

Up to 65 hours, the 20 minute defrost cycle came on every 4 hours. Beyond 65 hours, since the frost build up was relatively light, the 20 minute defrost cycle was programmed to come on once every 6 hours. In both cases, after the wall went through a few initial cycles, equilibrium in the temperature profile was always achieved in the cooling part of the cycle.

TABLE 20.1
INSULATION CONFIGURATION - SPECIMEN 3

| Test | Stud Space | 25 mm Cavity | Hat Section | Stud Cap |
|---|---|---|---|---|
| A | Yes | No | No | No |
| B | Yes | Yes | No | No |
| C | Yes | No | No | Yes |
| D | Yes | No | Yes | No |
| E | Yes | No | Yes | Yes |

Interior humidity was controlled within an average band of 39 to 44% RH. Air flow through the wall was set at 0.05 L/s.m$^2$ for the first 65 hours and then increased to 0.15 L/s.m$^2$ for the balance of the test.

At the end of the first 65 hours each of the 90 mm segments of rigid fibreglass insulation board were weighed. Board A which was on one side of the centre studs had gained a total of 6 grams of moisture while board B on the other side had gained 5 grams. Although moisture was not apparent on the insulation board surfaces, the bottoms of the boards did feel wet in the middle portion. Figure 20.9 is a sketch showing the bottom of the board along with the wet area. This observation provided evidence that condensation tended to drain through the open fibres of the insulation toward the bottom. There was no condensation apparent on any steel components.

At this point, the air flow rate was tripled to 0.15 L/s.m$^2$ for a further 55 hours. When the test was completed, the insulation boards were again weighed. Board A did not gain weight while Board B gained a further 4 grams of moisture. In this case, small frost beads were apparent on the exterior surface of Board B.

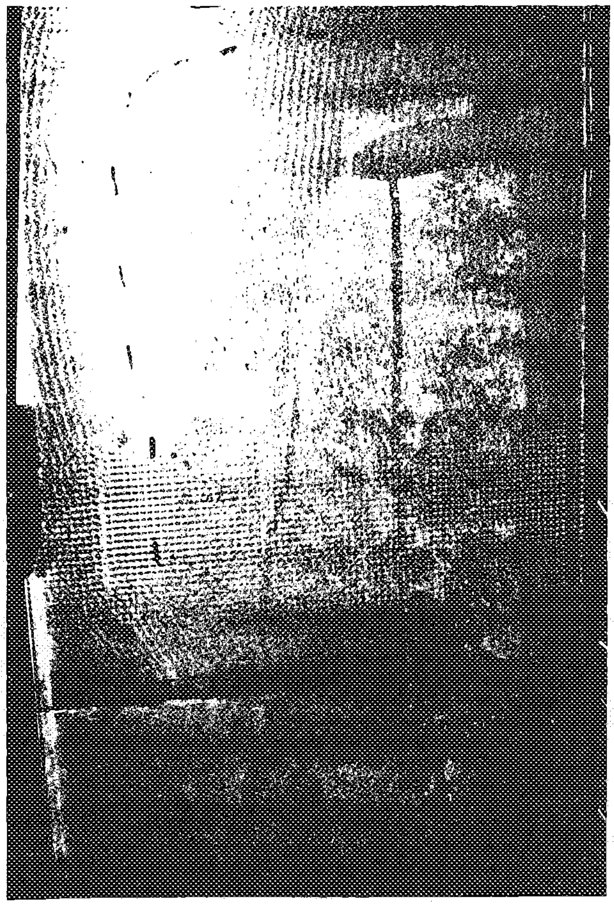

FIGURE 20.6 PHOTO SHOWING PATTERN OF MOISTURE ACCUMULATION (MOISTURE FORMED INSIDE DASHED OUTLINE AWAY FROM STUDS) - SPECIMEN 2A

FIGURE 20.7 PHOTO SHOWING FROST ACCUMULATION IN OPENING IN POLYSTYRENE SHEATHING

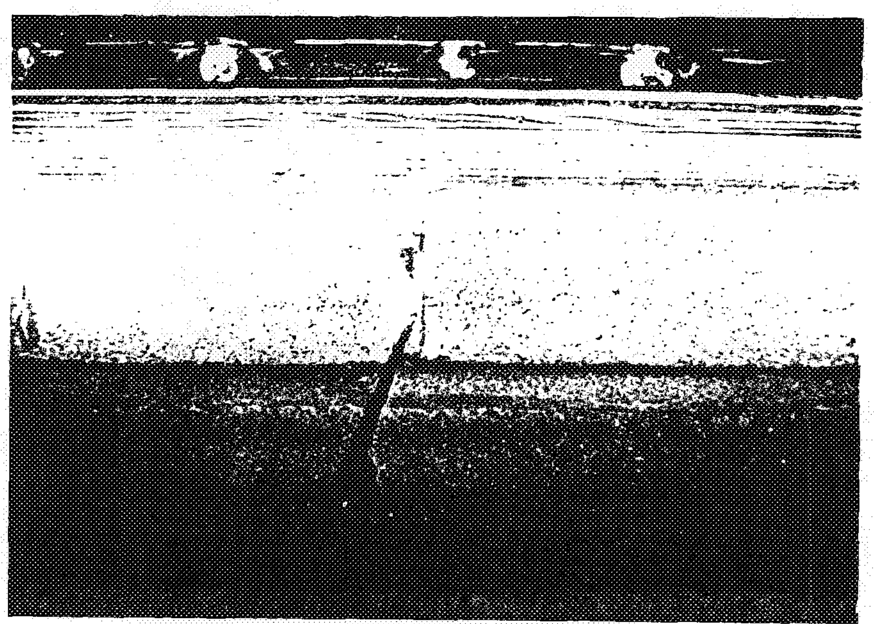

FIGURE 20.8 PHOTO SHOWING FROST ACCUMULATION IN VENEER VENTHOLES

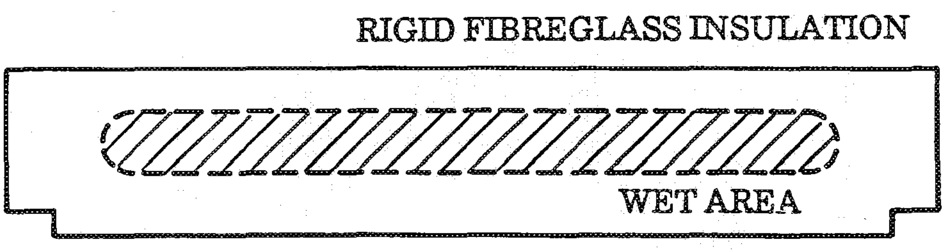


FIGURE 20.9 MOISTURE PATTERN AT BOTTOM OF INSULATION - SPECIMEN 3

## 20.6 DISCUSSION OF MOISTURE ACCUMULATION TEST RESULTS

### 20.6.1 Specimen 1A

The location of the corrosion, - on unplated fasteners, steel cuttings and steel burs - is of interest, since, although the fasteners can be richly plated and the stud can be heavily galvanized, the action of self-drilling a fastener into a stud invariably exposes unprotected steel. Bugle-shaped burs are of special significance since the deeper root, heavier gauge screws cause the biggest burs and thus expose the most steel. If corrosion begins to form at this point it could conceivably work its way in resulting in weakening of the screw/stud connection and possible failure.

The fact that the observed corrosion occurred in a period of 7 days with a very low flow rate of 0.015 L/s.m$^2$, indicates that it may be difficult to support the concept of a maximum allowable leakage value.

### 20.6.2 Specimen 1B

As in Specimen lA, air flowing through the wall specimen from interior to exterior resulted in the formation of corrosion products on unplated fasteners and on burs in fastener holes in the exterior stud face. This second similar result tends to validate the observations of Specimen 1A for the test conditions.

The relatively high air flow rate of 0.15 L/s.m$^2$ also resulted in moisture accumulation in gypsum board panels which indicates that exterior grade gypsum board can absorb condensation moisture on its back surface. This may be due to the fact that the glued paper joint in gypsum sheathing products can deteriorate with time and thus provide an entry point for water vapour. Since sheathing paper is manufactured to strict water tightness guidelines it is questionable whether water vapour actually diffused into the board.

### 20.6.3 Specimen 2A

Part 4 of the moisture accumulation testing of Specimen 2A is notable in that by giving an easy path for the leakage air to flow past the polystyrene, the path became a point of major moisture accumulation. The same phenomena was seen in the upper brick vent holes which also frosted shut. This would indicate that moisture tends to accumulate along the airflow path of least resistance. In Part 3 there was no single easy flow path and thus the pattern of accumulation was rather diffuse. This diffuse flow of air in Part 3 would be further supported by the finding in the air leakage testing of Specimen 2B that unpainted gypsum board is not air tight. Since the board of Specimen 2A was not painted, this would have been a source of diffuse air flow. On the other hand, in Part 4 the flow path was very distinct and was thus the main location for condensation.

### 20.6.4 Specimen 3

It has been stated that condensation tends to occur at the first surface below the dew point. In the moisture accumulation test of Specimen 3 this was not the case as indicated by the pattern of moisture on the bottom of the batt insulation (Figure 3.31). The fact that the inner part of the batt was wet does, however, substantiate the manufacturer's claim that this semi-rigid type fibreglass insulation is self draining. However, the claim that moisture in the batt does not affect thermal performance was brought into question by the test results as after ll7 hours of air flow under the test conditions, the thermal profile through the insulation was lowered slightly.

## 20.7 CONCLUSIONS AND RECOMMENDATIONS

### 20.7.1 GENERAL

The conclusions of this report are organized under the two headings, Experimental Apparatus and BV/SS Wall Performance. Recommendations as to the design and construction of BV/SS wall systems are also presented.

## 20.7.2 EXPERIMENTAL APPARATUS

Development of the Environmental Simulation Apparatus Test has shown that a single, compact test apparatus can be capable of evaluating the performance of wall systems under individually and simultaneously applied environmental loads. The cost advantage of testing small wall areas as opposed to full scale is significant and allows many more combinations of wall types and wall details to be investigated for the same cost. With ESTA, the systematic environmental evaluation of wall systems is brought into the realm of affordability. This should allow new wall systems to be thoroughly tested before being marketed. Current wall systems as well as new systems can be tested to identify weaknesses and provide suggestions for improvement.

## 20.7.3 BV/SS WALL PERFORMANCE

### 20.7.3.1 Air Leakage

- It has been demonstrated that even small openings in the air barrier can allow significant amounts of air leakage.
- Elements in the wall system other than the intended air barrier can act as partial barriers to air flow.
- Unpainted interior gypsum board is not air tight.
- Two coats of a latex paint on gypsum board is an efficient air barrier.
- Where an air tight seal is intended along any steel framing member, the heads of screws used to fasten the framing system together can introduce local air leakage paths.
- Electrical boxes can be made integral with an interior gypsum board air barrier.
- Even under laboratory conditions it was observed that air leaks paths can accidently occur in construction. Therefore design of air barriers should involve measurement of the perfection required during construction and the ability to correct faults after construction.
- Many air barriers can also act as vapour barriers and where this results in a double set of vapour barriers, condensation from exfiltrating air can be trapped leading to potential problems of wetting of sheathing and insulation and corrosion of steel components.

### 20.7.3.2 Thermal Performance

- Thermal bridges can result in cold spots along the inside surface of the wall.
- Air leakage paths around the insulation will allow cold exterior air to reduce the effectiveness of the insulation. This was particularly evident at stud locations where fit of the insulation was not always perfect.
- The steel framing portion of BV/SS wall systems can be kept above the dew point temperature for normal interior and climatic conditions.
- Many standard designs have parts of the wall below the dew point temperature of exfiltrating air.
- Two dimensional heat flow effects around studs spaced at 406 mm with the stud space insulated extended about 50 - 100 mm either side of the stud centerline.
- Large steel fasteners which by passed the insulating sheathing were shown to affect the local thermal profile by less than 5%.
- Any amount of air leakage can introduce the potential for moisture condensation at some point in the wall system.
- Even leakage rates as low as 0.03 $L/s.m^2$ were shown to result in significant accumulations of water.
- Condensation was shown to wet exterior grade gypsum board, batt insulation, and mutual components, depending on the design and arrangement of components.
- Where rigid insulation on the exterior of the steel studs acts as a vapour barrier, a sufficient thickness of this insulation will prevent condensation within the stud

cavity. Moisture will still condense along the leakage path (crack) through this insulation.

- Moisture accumulation due to vapour transmission was not measurable for the test performed.
- The size and distribution of leakage paths can cause condensation patterns to be either localized or diffused.
- Unplated screws as well as burs in the steel stud caused by screw application are areas where corrosion can initiate if they are below the dew point temperature.
- Keeping any component above the dew point will ensure that moisture will not condense at that point.

### 20.7.4 RECOMMENDATIONS

Air barriers should be included in the design of wall systems. However, generally it should not be assumed that an air barrier can be constructed in a perfectly air tight fashion. Allowance should be made for incidental leakage by examining the wall system to determine where condensation can form. Then the formation of such condensation should be assessed to determine if it will be detrimental to the serviceability of the wall system. For this reason it is suggested that air barriers placed toward the outside of the wall system be carefully evaluated. Any violation of the air barrier will be the point of moisture accumulation since air leakage and low temperatures will exist at such a location. This condensation may become trapped between the vapour barrier and the air barrier.

New designs of wall systems can be introduced to reduce the vulnerability of walls to effects of thermal bridges and condensation of moisture from exfiltrating air. It is recommended that designs be evaluated on the basis that some degree of imperfection will exist.

In the design of wall systems, including caulking, tape as well as major components, assessment of the long term performance of he wall should include provision for maintenance and repair.

### REFERENCES


1. ACHENBACH, P. R. "Design of a Calibrated Hot Box for Measuring the Heat, Air and Moisture Transfer of Composite Building Walls: ASHRAE SP28, Dec. 1979

2. ASTM, "C236 Steady State Thermal Performance By Means of a Guarded Hot Box", Annual Book of ASTM Standards, Philadelphia, 1988.

3. ASTM "C976 Thermal Performance of Building Assemblages by Means of a Calibrated Hot Box", Annual Book of ASTM Standards, Philadelphia, 1988.

4. ASTM "E283 Rate of Air Leakage Through Exterior Windows, Curtain Walls, and Doors", Annual Book of ASTM Standards, Philadelphia, 1988.

5. ASTM "E331 Water Penetration of Exterior Windows,Curtain Walls and Doors by Uniform Static Air Pressure Difference", Annual Book of ASTM Standards, Philadelphia, 1988.

6. COWIE ENGINEERING LIMITED, "Investigation of Exterior Walls, Windows and Roof Alderney Manor, Alderney Drive, Dartmouth, Nova Scotia", Report for Central Mortgage and Housing Corporation, Halifax Office, 1976.

7. GRIMM, C.T., "Brick Veneer: A Second Opinion", The Construction Specifier, Alexandria, Vt., April, 1984.

8. HANDEGORD, G.O.P., "The Need for Improved Air Tightness in Buildings", Building Research Note No. 151, DBR NRCC, November 1979.

9. HUTCHEON, N.B., "Humidity in Canadian Buildings", CBD 1, DBR, NRC, Ottawa, January 1960, pp.4.

10. HUTCHEON, N.B., "Humidified Buildings", CBD 42, DBR, NRC, Ottawa, June 1963, pp 4.

11. HUTCHEON, N.B., and HANDEGORD, G.O.P., Building Science for a Cold Climate, John Wily & Sons, Toronto, 1983, 440 pages

12. KLUGE, A., and DRYSDALE, R.G., "Environmental Performance of Brick Veneer Curtain Walls - Background and Description of a Test Apparatus", Fourth North American Masonry Conference, Los Angeles, 1987.

13. LATTA, J.K., and BEACH, R.K., "Vapour Diffusion and Condensation", CBD 57, DBR, NRCC, September 1964.

14. MASONRY COUNCIL OF CANADA, "Guide to Energy Efficiency in Masonry and Concrete Buildings", 1982, 144 pages.

15. ASSOCIATE COMMITTEE ON THE NATIONAL BUILDING CODE, "National Building Code of Canada", Ottawa 1985.

16. QUIROUETTE, R.L., "The Difference Between a Vapour Barrier and an Air Barrier", Building Practice Note No. 54, DBR/NRCC, July 1985

17. ROUSSEAU, M.Z., "Control of Surface and Concealed Condensation", Building Science Insight '83, DBR/NRCC, May 1984

18. SASAKI, J.R., "Thermal Performance of Exterior Steel Stud Frame Walls", NRCC Publication No. 2226, DBR, 1972.

19. SOLVASON, K.R., "Large-Scale Wall Heat-Flow Measuring Apparatus", ASHRAE Transactions, 65, 1959

20. WILSON, A.G., and GARDEN, G.K., "Moisture Accumulation in Walls Due to Air Leakage", DBR Technical Paper No.227, NRCC, 1965

21. WILSON, A.G. and TAMURA, G.T., "Stack Effect in Buildings", Canadian Building Digest, No. 104, Building Research, National Research Council, Ottawa, 1968.

22. WILSON, A.G., "Air Leakage in Buildings", Canadian Building Digest No. 23, NRCC, 1961.

# CHAPTER 21

## A SUMMARY OF

## BEHAVIOUR OF BRICK VENEER/STEEL STUD TIE SYSTEMS

by

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and

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## 21.1 INTRODUCTION

This chapter is a summary of research information on the strength and stiffness characteristics of wall ties currently used to attach brick veneer to steel stud backup walls. The reported research is Part 4 of the five part McMaster University Laboratory Test Program on BV/SS Wall Systems sponsored by Canada Mortgage and Housing Corporation. Use of steel studs as the backup wall for brick veneer wall systems followed the earlier development of brick veneer supported by concrete block backup for both low and high rise construction. It also had obvious similarities to low rise residential use of wood stud walls as backup for the brick veneer. Since widespread use of the BV/SS form of construction preceeded research, development of accepted standards for construction, dissemination of educational information to the building profession, or even significant field performance experience, it is not surprising that much of the design and construction practices were borrowed from the previously mentioned wood stud or concrete masonry backup walls.

The above is particularly true in the area of design and construction practices related to BV/SS ties. For this reason, it was relatively common place to find 28 gauge corrugated strip tie (previously allowed for low rise residential housing) used on multi-storey construction without regard to loading conditions or corrosive atmosphere[9]. In addition, neither the design codes nor individual designers made any distinction between spacing requirements for ties regardless of the type of backup wall. In fact, it is only very recently that theoretical and experimental research has confirmed earlier calculations which showed that for BV/SS construction, top ties or ties near cracks in the veneer can be subjected to forces approaching half the lateral load on the wall.

Coincident with the introduction of BV/SS wall systems into the construction scene was a major growth of interest in the building science related issues for building envelopes. Deterioration of walls due to moisture resulting from rain penetration, water vapour transmission through materials, and air borne moisture from exfiltrating air was identified as a serious problem both from the view point of safety as well as protection of investment. In addition, variable thermal performance, drafts and staining were identified as unacceptable. As a result of these considerations, design criteria, construction practices and introduction of new materials have all combined to alter the requirements for BV/SS ties.

Briefly, it was determined that the adequacy of ties for BV/SS construction should be judged in terms of the following:

- Adequate strength and stiffness for structural requirements.
- Sufficient robustness and durability to tolerate job site conditions with damage.
- Either tolerance for or designed in limitations on adjustability, variations in installation locations, or misalignment so that specified properties can be relied upon.
- Adequate corrosion protection, including attachment mechanisms, to maintain structural integrity for the life of the structure.
- Compatibility of construction requirements with the design and installation of exterior sheathing, insulation, vapour barriers, and air barriers.
- Cost and convenience of use including level of inspection required.

Laboratory tests of tie systems, performance criteria as well as actual field performance have been reported by several bodies including The Brick Institute of America[2,3,4,5], the University of Alberta[24,20,18], Building Research Establishment[25,11,12], and others[1,9,10,15,16,17,26]. Also, two standard codes exist which specifically address masonry ties[27,8].

## 21.2 TIE SYSTEMS SELECTED FOR TESTING

One of the most continuously changing and ever developing components of the BV/SS system is the system used to attach the brick veneer to the backup wall. In the past, the main design requirement was that these tie systems must transfer the lateral wind induced loads from the brick veneer to the steel stud backup wall. However the need to accommodate inevitable construction tolerances, the increased awareness of substantially greater structural requirements for individual ties, and impinging building science concerns have had the combined effect of leading to development of ties designed specifically to satisfy some or all of a new set of requirements. While performance standards for BV/SS tie systems do not yet exist, some general requirements which have gradually become accepted include limited mechanical movement or "play", minimum stiffness and capacity, and non-destructive interference with air and vapour barriers.

Twelve tie systems were selected for this test program. (Ties are illustrated in Figure 21.4 along with performance curves). It should be noted that for this research program, full co-operation was received from the tie manufacturers and that all of the ties were donated by the following companies: Bailey Metals Products Limited, Blok-Lok Limited, Dur-O-Wal Inc., Fero Holdings Limited, Hohmann and Barnard Inc. and Thermosteel Building Systems Inc. Catalog or serial numbers are not used to identify the different ties. Instead, an easily identified trade or generic name is used.

## 21.3 DESCRIPTION OF THE TEST APPARATUS

The test apparatus used in the program was developed primarily to provide a standardized test that can be used to study the behaviour of the many different tie specimens prepared for testing. The test apparatus is illustrated in Figure 21.1. A requirement was to be able to define the strength and stiffness characteristics under various conditions of attachment and adjustibility. Thus an important feature of the apparatus was to have flexibility in setup without jeopardizing any consistency in the test procedure.

In Figure 21.1, two clamping devices are shown. The wire clamping devices served the function of the mortar joint in the veneer while the other secured the 450 mm long stud specimen to the loading table. Linear variable differential transducers (LVDT) and a load cell were employed to measure the displacements and applied loading during the tests.

The recorded data included the applied load, global displacement and relative displacement between the stud centreline and the loading table. The latter displacement was then subtracted from the global displacement to determine the cavity displacement, which is independent of any beam deflection and represents tie movement, local stud deformation, fastener slippage and flange rotation. The test frame was a MTS Testing Machine operated under controlled displacements and constant speed. Each of the components of the test apparatus are described in greater detail below.

**Cavity Width:** This term is used to describe the unsupported length of the wall tie. Where the tie is directly supported on the steel stud, the sheathing is assumed not to offer any significant support to the tie. For ties mounted on exterior sheathing, the cavity width is the clear distance between the sheathing and the veneer.

**Position A, B & C:** These terms refer to the position of the tie on the flange of the stud with respect to the web. As shown in Figure 21.2, position A is 15 mm from the web (approximately the middle), position B is 10 mm and C is 20 mm. These three locations on the stud flange were used to represent the range of tie attachment positions.

**Mechanical Play:** The movement in the tie system before it is able to resist loading is defined as the mechanical play. For example the difference between the size of a hole and the diameter of the wire pintle is the mechanical play.

**Tie Clamping Device:** Owing in part to the large number of test repetitions planned for this test program, it was necessary to forgo the construction of brick assemblages as part of the test specimen. Instead, a clamping device was designed to replace the role of the mortar joint between bricks where the wire tie would be located. Since many of the wire ties are bent or formed in some pattern to support the ties, it was necessary to supply a template to fit snugly within the tie. The thickness of the template was made less than the wire diameter to allow the clamping angles shown in Figure 21.3 to bear directly against the wire and thus provide restraint to any movement within the clamping devices during the test. The template, as illustrated in Figure 21.3, was intended to act in a similar manner to an actual mortar joint and prevent the tie from straightening out during a tension test.

The clamping device itself consisted of two steel angles placed back to back. One angle was permanently fastened to the load cell plate to ensure that all the specimens were placed into the apparatus concentric with the load cell. The other angle was fitted with a slotted hole to allow fastening and unfastening of the test specimens. Load-displacement tests conducted on wire ties in the clamping device confirmed that any slippage under the expected loading range was so small that it was not detectable. Therefore cavity displacement measurements did not include any component of pull out of the tie from the (modelled) mortar joint.

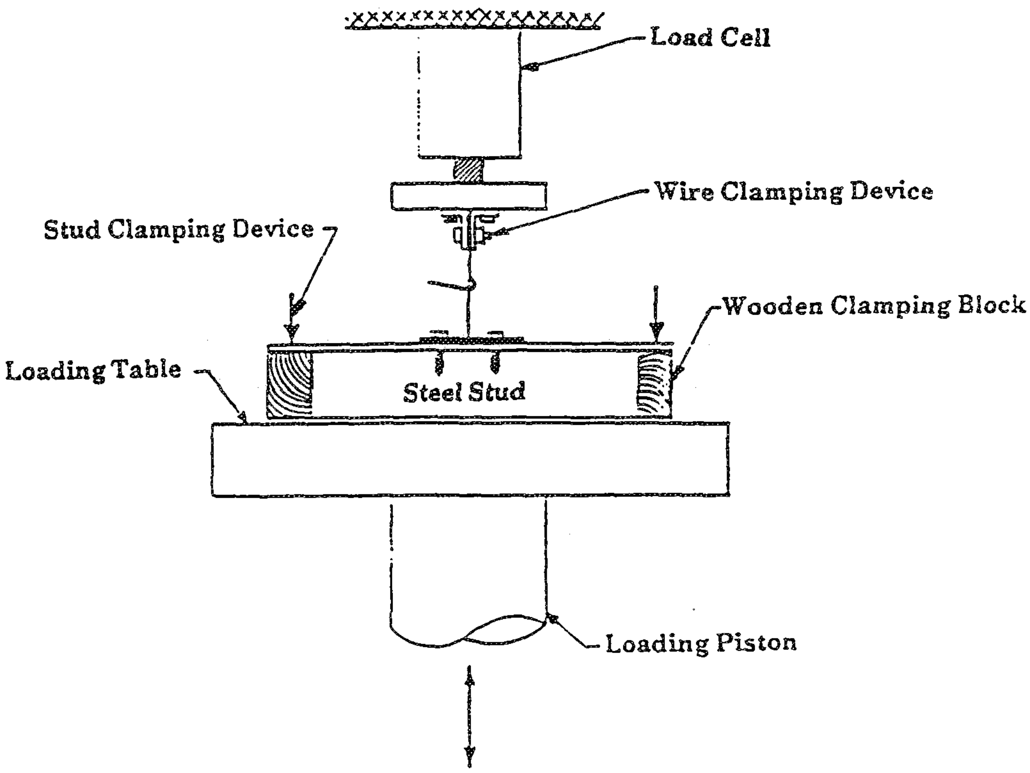


FIGURE 21.1 TEST APPARATUS

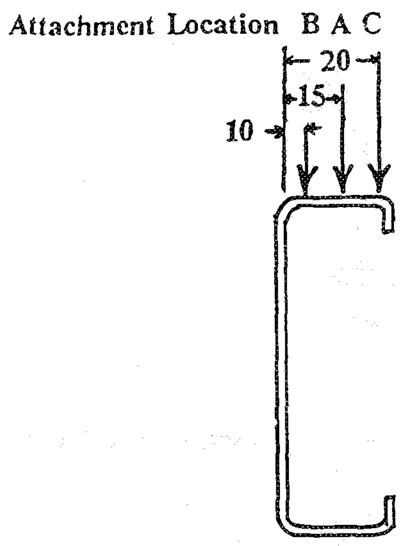


**FIGURE 21.2 ILLUSTRATION OF THE ATTACHMENT POSITIONS ACROSS THE FLANGE OF A STEEL STUD**

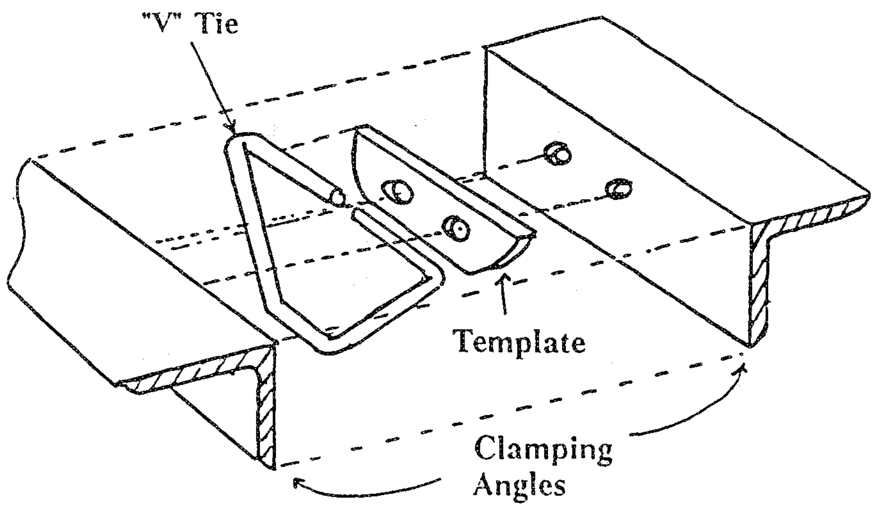


**FIGURE 21.3 TIE CLAMPING DEVICE**

**Stud Specimen Clamping Device:** The tie clamping device secured the tie portion of the BV/SS tie system in a place where it could be fastened to the anchor portion already attached to the stud specimen. Once the entire unit was placed in the test apparatus and secured to the overhead load cell, it could be attached to the loading table.

The stud specimen was clamped at its ends where wood inserts were used to prevent the stud from being deformed by the firmly placed clamps. Since the stud rested against the loading table along its bottom flange, no beam deflection would occur under compression load on the tie. Thus for tie compression tests, the recorded displacement included tie induced flange rotation, local dimpling of the stud and any axial deformation in the tie unit (including compression of sheathing if present).

For the tie tension tests, which produced upward deflection of the stud, the beam deflection component of displacement had to be measured so that it could be subtracted from the overall displacement to give the cavity displacement. This simply involved subtracting the measured movement between the loading table and the stud's centre line from the global displacement. Although the stud deflection was found to be rather small, for completeness it was subtracted to give the cavity displacement.

**Displacement Measurement Devices:** Linear variable differential transducers (LVDT) were employed to measure the displacements. The measurements were recorded as voltages by a Hewlett Packard scanner and converted to units of length using the appropriate calibration curve. These calibration curves were established in the laboratory and continuously monitored for accuracy.

**Load Monitoring Method:** The load cell was located between the head and the table of the MTS loading machine and functioned in both compression and tension. The load cell sent a voltage signal to the scanner which was then converted to units of force using the appropriate calibration constant.

**Test Machine:** The test frame used in the program was a MTS (Material Testing Machine). The machine operated under controlled displacements with manually controlled speed. The speed was kept constant for the duration of the test. Since the load cell was constantly monitored by the scanner, pauses in the tests could be introduced at relatively constant load increments so that all the transducers' signals could be permanently recorded by the data scanner.

## 21.4 DESCRIPTION OF TEST SERIES

For each type of tie, several different types of tests were performed in order to determine the influence of various factors on tension and compression strength and stiffness characteristics. Depending on the nature of the tie, different combinations of tests were performed for each type of tie. To avoid having to repeat the descriptions of the individual types of tests, a complete listing of all the types of tests is provided below where identification numbers were assigned to each series. In the identification number, T stands for tie and the actual tie number (1 to 12) is used in place of the question mark. The final number is used to indicate the type of test as described below: (Note - The following represents only a portion of the complete test program)

T-?-1 Compression test conducted on a tie system mounted at position A (middle of the flange) on a 20 gauge steel stud.

T-?-2 Tension test conducted on a tie system mounted at position A on a 20 gauge steel stud.

T-?-3 Tension test conducted on a tie system mounted at position A, B and C on a 20 gauge steel stud. (This series of tests was used to investigate the influence of attachment location on tie performance).

T-?-4 Tension test conducted on a tie system mounted at position A on a 20 gauge steel stud. (This series of tests was used to investigate the influence of adjustability of ties on performance).

T-?-5 Compression test conducted on a tie unit mounted on a non-yielding support. (This series of tests was used to investigate the influence of cavity width without the influence of the steel stud).

T-?-6 Tension test conducted on a tie system mounted at position A on an 18 gauge steel stud. (This series of tests was used to investigate the influence of steel gauge on tie performance).

T-?-7 Compression test conducted on a tie unit mounted on a non-yielding support.

T-?-8 Tension test conducted on a tie unit mounted on a non-yielding support.

## 21.5 TEST PROGRAM RESULTS

### 21.5.1 GENERAL

The experimental program includes testing of the complete tie system including interaction with the steel stud and sheathing materials. Also the characteristics of the tie itself were determined by testing the tie alone mounted on a rigid steel section. For the latter tests, bolts with diameter and head sizes comparable to the self drilling screws were employed. These tests correspond to the Test Series T-?-1,2,7 and 8.

In this section, the data from tests of each of the ties is presented separately to illustrate its characteristic load-displacement behaviours as established in the laboratory. The graphs represent the performance domain of the mean compression and tension curves for ties attached to the steel stud and separately attached to the rigid supports. For comparison between the various ties tested, figures have been provided to illustate different observed behaviour characteristics.

For the standard tests, 20 gauge 90 mm deep steel studs were used exclusively. To serve as a comparison, a later section contains the experimental findings for tests completed on 18 gauge studs. For standard tests, the ties were fastened to the flange of the stud at a distance of 15 mm away from the web. This was considered to represent the average location of attachment. The influence that the location of attachment had on tie performance is also presented in a later section.

For convenience during discussions, an attempt was made to classify the adjustable wall ties. Four distinct classes based on adjustable features can be described as follows:

Class 1 - A slot in the anchor accommodates various positions of the wire tie extending from the veneer.
Class 2 - An extended vertical portion of the brick wire tie allows adjustment.
Class 3 - Movement of the anchor portion, attached to the steel stud permits vertical adjustment relative to the fastener.
Class 4 - Either non-adjustable or having a feature permitting alternate locations of parts of the tie.

The twelve types of ties included in this test program were classified based on the above criteria and they are listed in Table 21.1 according to these classifications.

### 21.5.2 GENERAL PERFORMANCE OF INDIVIDUAL TIES

Table 21.2 contains a summary of the mean results for the different series of tests completed to indicate the general characteristics of each type of tie. In the following section, specific comments related to the general performance of each tie are provided in turn. These results were for Test Series T-?-1, 2, 7 and 8. Comparison between ties can be made with the aid of Figure 21.4.

*** T-1: The Double Leg Adjustable Tie (DLA)**

From the curves in Figure 21.4a, it can be observed that, although attachment to the steel stud does not greatly alter the general behaviour characteristics, it does have a pronounced effect on the stiffness of the system. From the tests on the steel stud, the tension behaviour is reasonably linear compared to the non-linear behaviour for the compression tests. This behaviour is repeated in the tests on the tie unit itself. The difference in stiffness between the two compression tests can be attributed to the flange rotation experienced by the specimen mounted on the steel stud. In tension the stud mounted specimen exhibited flange rotation and eventually experienced screw pull out. In compression the failure was characterized by sideways deflection of the anchor legs followed by buckling of the wire tie. Since the two legs do not share the load equally (because of flange rotation), failure was initiated in the most heavily loaded leg; being closest to the stud web.

## TABLE 21.1 SUMMARY OF WALL TIE PROPERTIES

| | Adjustable Range (mm) | Mechanical Play (mm) | Anchor Yield (MPa) | Cov. (%) | Wire Yield (MPa) | Cov. (%) |
|---|---|---|---|---|---|---|
| CLASS 1 | | | | | | |
| Wrap Around Tie (WAT) | 76 | 0.75 | 353 | 0.3 | 615 | 0.8 |
| 'C' Type Anchor (CTA) | 90 | 1.24 | 287 | 0.3 | 615 | 0.8 |
| D-W 10X Wall Tie (DW10) | 90 | 0.3 | 346 | 1.7 | 615 | 0.8 |
| CLASS 2 | | | | | | |
| Double Leg Adjustable (DLA) | 35 | 1.24 | 283 | 0.3 | 696 | 8.1 |
| Flange Locking Anchor (FLA) | 35 | 0.3 | 357 | 1.5 | 490 | 1.7 |
| Single Leg Adjustable (SLA) | 35 | 0.2 | 292 | 4.1 | 490 | 1.7 |
| CLASS 3 | | | | | | |
| Speed Set Anchor (SSA) | 35 | - | 429 | 4.4 | - | - |
| Wire Loop Adjustable (WLA) | | 35 - | - | - | 613 | 0.6 |
| CLASS 4 | | | | | | |
| Self-Drilling Tie (SDT) | 4.8 | 0.9 | - | - | 615 | 0.8 |
| Corrugated Strip Tie (CST) | - | - | - | - | - | - |
| Brick Veneer Tie Support (BVTS) | - | - | - | - | - | - |
| Thermosteel Anchor (TA) | 33 | 1.6 | 418 | 1.0 | 615 | 0.8 |

## TABLE 21.2 SUMMARY OF TIE TEST RESULTS

| TIE | TEST[2] | ULT. LOAD (N) | COV. (%) | LOAD @ 1.2 mm (N) |
|---|---|---|---|---|
| DLA | T-1-1 | 1678.0 | 8.1 | 730 |
| | T-1-2 | 1708.9 | 10.9 | 436 |
| | T-1-7[xx] | | | 1732 |
| | T-1-8 | | | 774 |
| CTA | T-2-1 | 2278.3 | 1.9 | 826 |
| | T-2-2 | 2190.6 | 1.8 | 104 |
| | T-2-8 | | | 326 |
| SSA | T-3-1 | | | 888 |
| | T-3-2 | 1167.5 | 3.6 | 582 |
| | T-3-7 | | | 2506 |
| | T-3-8 | | | 2082 |
| SLA | T-4-1 | 1087.5 | 6.5 | 644 |
| | T-4-2 | 1004.5 | 7.0 | 245 |
| | T-4-7 | | | 1093 |
| | T-4-8 | | | 682 |
| WLA | T-5-1 | 2457.6 | 1.4 | 718 |
| | T-5-2 | 1287.5 | 4.6 | 484 |
| | T-5-7 | | | 2475 |
| | T-5-8 | | | 1498 |
| FLA | T-6-1 | 1760.7 | 3.9 | 570 |
| | T-6-2 | 1128.7 | 6.1 | 505 |
| | T-6-7 | | | 1014 |
| | T-6-8 | | | 1120 |
| WAT | T-7-1 | 4500.4 | 2.0 | 930 |
| | T-7-2 | 2641.8 | 4.6 | 829 |
| SDT | T-8-1 | | | 626 |
| | T-8-2 | 1417.9 | 3.1 | 635 |
| DW10 | T-9-1[xx] | | | 372 |
| | T-9-2 | 2007.3 | 3.4 | 328 |
| | T-9-8 | | | 640 |
| CST | T-10-1 | 699.0 | 6.3 | 551 |
| | T-10-2 | 1481.0 | 3.0 | 108 |
| TA | T-12-1[xx] | 3514.9 | 13.7 | 1515 |
| | T-12-2[xx] | 4107.6 | 10.8 | 1055 |

2. T-?-1 = compression test of system
   T-?-2 = tension test of system
   T-?-7 = compression test of tie alone
   T-?-8 = tension test of tie alone

3. 25 mm cavity space used.

## TABLE 21.3

## SUMMARY OF INFLUENCE OF LOCATION

| TIE | TEST[2] | ULT. LOAD (N) | COV. (%) | LOAD @ 1.2 mm (N) |
|---|---|---|---|---|
| DLA | T-1-3A | 1723.1 | 9.6 | 523 |
| | T-1-2 | 1708.9 | 10.9 | 436 |
| | T-1-3B | 1620.5 | 2.4 | 236 |
| CTA | T-2-3A | 2125.5 | 4.7 | 107 |
| | T-2-2 | 2190.6 | 1.8 | 104 |
| | T-2-3B | 2155.3 | 2.6 | 110 |
| SDT | T-8-3A | 1559.7 | 2.8 | 1055 |
| | T-8-2 | 1417.9 | 3.1 | 635 |
| | T-8-3B | 1373.8 | 3.9 | 400 |

2. Distance of attachment from outside face of web of stud.

3A - 10 mm
2 - 15 mm
3B - 18 mm

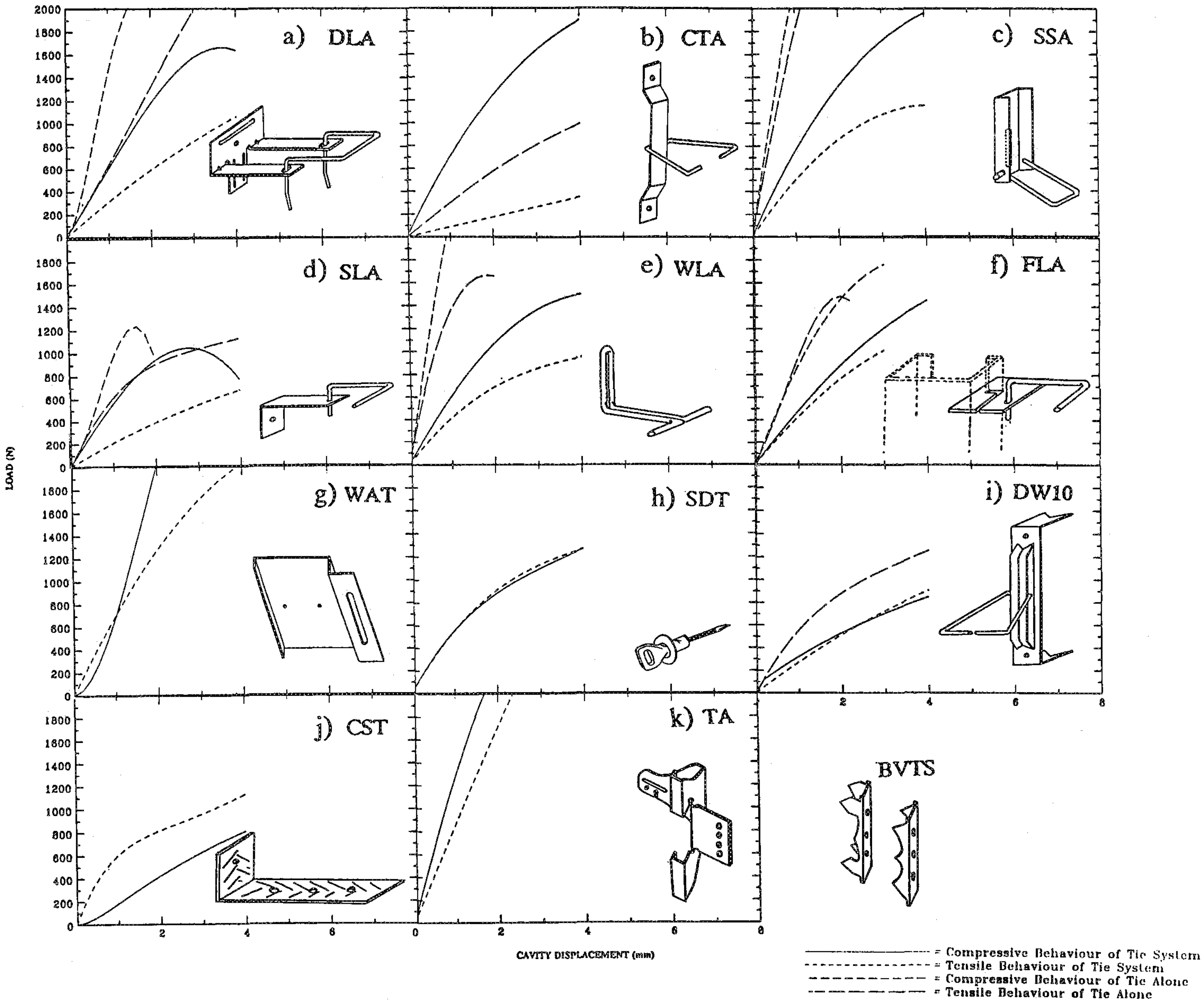


FIGURE 21.4 GENERAL PERFORMANCE OF INDIVIDUAL TIES

* **T-2: "C" Type Anchor (CTA)**

The compression and tension test curves shown in Figure 21.4b graphically demonstrate the recorded large differences between the stiffnesses under compressive and tensile loading. In tension the tie was very flexible owing to the fact that it was loaded in the middle of its adjustable range. This configuration proved to be the most flexibility for this particular tie.

* **T-3: Speed Set Anchor (SSA)**

This tie was mounted on 12mm exterior grade gypsum board sheathing and the self-drilling screw was fastened such that 15 mm separated the wire tie from the screw position on the anchor. Because of the eccentricity of loading on the wire part of the tie, it was observed that as the tie was either pushed into the gypsum board in compression or pulled away in tension, a prying action existed on the screw. Additional tests presented in the complete report provide a more detailed investigation of the influence of this prying action. The general tie performance is seen if Figure 21.4c.

* **T-4: The Single Leg Adjustable Tie (SLA)**

From the compression and tension test curves shown in Figure 21.4d, it can be observed that, although the influence of the steel stud did not effect the general behaviour characteristics of this tie, it had a pronounced effect on the stiffness of the system. This can be attributed to the flange rotation experienced by the specimen mounted on the steel stud and, for tension, by the pull-out of the screw. When the tie was tested in compression on the steel stud there was bending observed in both the anchor leg and the wire pintle. The bending was most prominent at the 90 degree bend in the anchor and at the clamped edge in the wire.

In the tension tests, the tie anchor and wire tended to straighten, but before the wire could slip through the hole in the anchor, the screw pulled loose. In all the tension tests screw failure prevailed. The loading on the screw was not strictly axial since the anchor tended to pry on the screw and cause a shearing force.

* **T-5: Wire Loop Adjustable Tie (WLA)**

In Figure 21.4e, the difference between the compression and tension tests with the tie attached to the stud (and mounted on gypsum board sheathing) and those with the tie attached to the rigid support indicated that the steel stud and sheathing strongly influenced both the failure and stiffness characteristics of the wire loop tie. This can be attributed to its one piece construction. Other ties that consist of two parts usually possess a weak link which is more flexible than the stud flange. In essence this tie merely acts as a load point on the flange of the stud. Thus it is not surprising that the two tests produced quite distinct behaviour characteristics.

* **T-6: Flange Locking Anchor Tie (FLA)**

When tested in compression on both the steel stud and the rigid support, the anchor first twisted sideways and then continued to buckle at both the flange connection and along the unsupported length of the wire pintle. In tension, the tests consistently revealed that the anchor itself yielded prior to the wire pintle. Failure was observed as a continuous opening up of the anchor at the slot accommodating attachment to the flange of the stud. For the tests conducted on steel stud, this was accompanied by local bending and rotation of the stud's flange since the load was ultimately transferred at the outer edge of the flange itself. As can be seen in Figure 21.4f, the transfer of both compressive or tensile load to the stud resulted in significant loss of strength and stiffness.

* **T-7: Wrap Around Tie (WAT)**

Although the compression behaviour of this tie eventually became stiffer than for tension, initially its stiffness was greatest under tension loading. This flexibility in compression can be attributed to some initial bending in the flange of the tie itself. The flange did not perfectly fit the profile of the steel stud and when first loaded there was movement until the tie and stud flanges were in more complete contact. Ultimately, however, it was the wire tie that yielded first. The tension tests were carried out in the middle or neutral position of the adjustable range. Later in the report it will be shown that this location actually provides the least stiffness. Ultimately the metal strip along the outside of the adjustable slot twisted and bent, causing a loss of stiffness in the tie system. The general performance of the tie is seen in Figure 21.4g.

* **T-8: Self-Drilling Tie (SDT)**

For this self-drilling tie, the compression test was terminated after significant deformation in both the steel stud and the wire tie. No noticeable deformation was observed in the tie itself. When tested in tension the screw was pulled from the steel stud. In a later section of this report, the influence of securing the screw tie with a nut on the inside of the flange will be reported. The general performance of the tie is seen in Figure 21.4h.

*** T-9: The DW-10X Tie**

The DW-10X tie was mounted on 38 mm (1.5 in.) Styrofoam SM board. The compression test of the tie system revealed that the anchor yielded and deformed into the SM board. When tested in tension, again it was the tie anchor that yielded. It should be noted that the tie was set in the middle of the adjustable range. This tie position was found to be neither the stiffest configuration for the tie nor the most flexible. The general performance of the tie is seen in Figure 21.4i.

*** T-10: The Corrugated Strip Tie (CST)**

When the corrugated strip tie was tested in tension, it pulled up straight and eventually the screw was pulled from the stud. In compression the tie buckled. These tests were completed on an unsupported tie length of 1 inch and represent a lower bound for the performance of this tie since larger cavity widths are not allowed[8]. In the complete report, the results from other tests incorporating some of the many variables that are consistent with use of this type of tie in the field will be presented. The load-deflection data was plotted in Figure 21.4j.

*** T-11: The Brick Veneer Tie Support (BVTS)**

Since the characteristics of the Brick Veneer Tie Support are very much influenced by the type of tie attached to it, results are not presented in this general section.

*** T-12: Thermosteel Tie (TA)**

This tie was unique in that it requires a special type of steel stud for its application. This caused some difficulty when comparing this tie's performance with others.
From the graphs in Figure 21.4k, it can be observed that the compression-tension behaviours are unique. The tie was stiffest when loaded in compression. When the tie was tested in compression the wire tie ultimately bent at the tie anchor connection and in tension the tie became disengaged from the thermosteel stud. It was observed that the clip, which was bent around the flange/chord of the stud to secure the tie on the stud, gradually came free from the stud under tension loading.

### 21.5.3 INFLUENCE OF LOCATION OF ATTACHMENT OF TIES

Brick veneer steel stud wall ties installed in the field are vulnerable to variable attachment locations on the steel studs. Ideally, from an inspection point of view, it is most advantageous to have the tie mounted directly on the stud where its location can be inspected. Unfortunately this is impractical for some tie types owing to the extent of puncturing of the sheathing and inherent damage to the air and vapour barrier designs which would be necessary. Thus many ties are fastened directly on the sheathing. Such ties make it very difficult to either determine or control just where on the flange of the stud they are fastened. For this reason, the test program included an investigation on the influence of attachment location on tie performance. A summary of the results for this influence were listed in Table 21.3. These test series were limited to tension tests and basically provided stiffness information for ties tested on 20 gauge steel studs fastened at three locations on the flange. The additional tests for this comparison correspond to test series T-?-3 of the test program.

The three ties used to investigate the influence of attachment location included: T-1, the double leg adjustable tie (DLA), T-2, the 'C' type tie (CTA), and T-8, the self-drilling tie (SDT). The figures provided for comparison purposes were limited to the standard performance domain of 4 mm to avoid confusion or misinterpretations which could result from observing the full range of deformations.

When the DLA and SDT were tested at the three locations of 10, 15 and 20 mm from the web, considerable differences in stiffness were observed. However when the CTA was tested it was not initially obvious why very little influence was observed. These effects can be seen in Figure 21.5a, 5b and 5c.

An explanation is that the SDT and the DLA were comparatively stiff and thus the rotation of the flange of the stud significantly contributed to the displacement. Thus flange rotation displacement and correspondingly the cavity displacement increased with increased attachment distance from the web. Conversely with the CTA, the tie itself was more flexible than the flange and in fact,

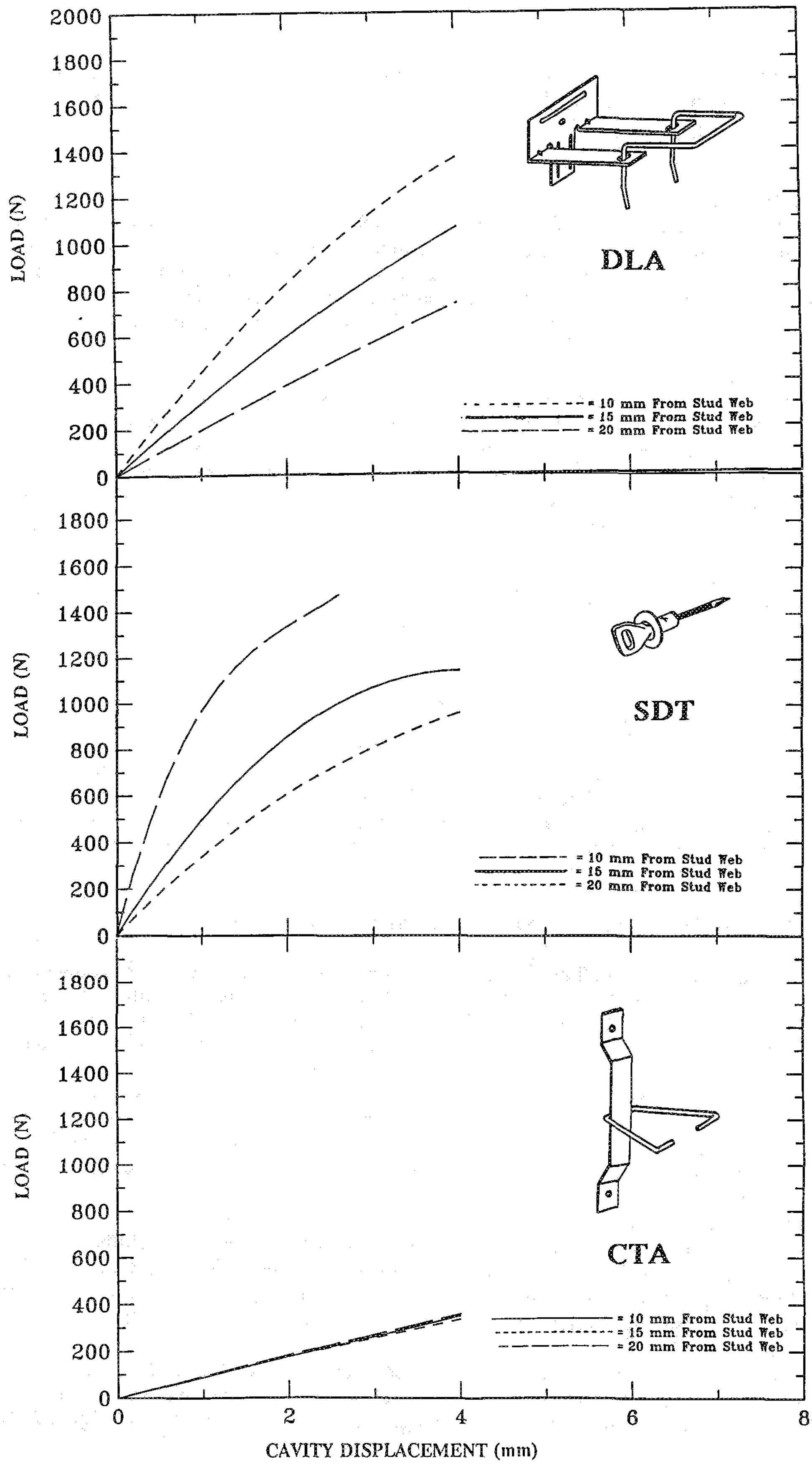


**FIGURE 21.5 INFLUENCE OF ATTACHMENT LOCATION**

it tended to brace the flange of the stud. Hence, the greatest portion of the cavity displacement was from tie deformation which was relatively independent of attachment location.

### 21.5.4 INFLUENCE OF RANGE OF ADJUSTMENT ON TIE PERFORMANCE

To accommodate field installation tolerances, most brick veneer steel stud wall ties provide for some degree of adjustment. In this report, the adjustable ranges over which the ties were intended to function in a satisfactory fashion were measured and reported in Table 21.1. A separate test series summarized in Table 21.4 was dedicated to studying the influence of the adjustable ranges on the performance of various ties. Figure 21.6 was drawn to illustrate the adjustable range for each of the included ties.

This portion of the test program investigated the changes in tie strength and changes in stiffness within the performance domain at different degrees of adjustment. All ties were tested at the lower, upper and intermediate levels of their adjustable ranges. The tests were carried out in tension on 20 gauge steel studs with the tie fastened to the flange of the stud 15 mm from the web. Once again the load-displacement curves were limited to the performance domain of 4 mm of cavity displacement. These tests correspond to test series T-?-4 of the test program.

* **T-1: Double Leg Adjustable Tie (DLA)**

This tie can be adjusted in the field by sliding the double leg pintle up and down through slotted holes in the anchor legs. The upper bound on the adjustable range was set at 35 mm. For this tie, the lower bound occurred when the bent part of the wire pintle and the legs of tie anchor were positioned tight together.

Larger adjustments translated into increased moment arm on the wire pintle, as illustrated in Figure 21.6. This increased bending caused the wire pintle to yield earlier and as shown in Figure 21.7a, test results showed that as the adjustment was increased the system became more flexible and the capacity at 4 mm displacement and at failure decreased.

* **T-2: C Type Tie (CTA)**

The adjustment of the 'C' tie lies in the ability of the triangular wire tie to move between the anchor and the stud or sheathing. For this tie, tests were performed at the middle, quarter and end points of the adjustable range. The response shown in Figure 21.7b for these loading positions provided some interesting observations. When the tie was tested in the middle position, the anchor yielded and bent upwards and, after considerable displacement, pulled the screw free in a shearing manner. When the test was conducted at the quarter point the displacements decreased and the screw was pulled out in more of a direct tension fashion. However, when the tie was tested at the end point, the combination of localized axial pull and shear force on the screw provided the system with a greater stiffness and increased the overall capacity of the tie system.

* **T-3: Speed Set Anchor (SSA)**

The adjustment of the speed set anchor relied on the anchor itself moving under the fastening screw already attached to the stud and sheathing. For these tests, 12 mm gypsum board was employed as the sheathing. The lower bound of the adjustment range was identified when the fastening screw was 10 mm from the wire tie. The upper bound was at 25 mm.

The test results, shown in Figure 21.7c, indicated that the lower bound provided the most flexibility. It was observed that this position most nearly induced direct axial load on the screw which in turn resulted in large displacements. The greatest stiffness was achieved when the screw was located in the middle of the adjustment range. Similar to the CTA, the interaction of axial and shearing load produced the stiffest tie system. For the upper bound of the adjustment range, it was observed that the anchor first pried then pulled the screw free.

* **T-5: The Wire Loop Adjustable Tie (WLA)**

In a similar fashion to the SSA, adjustment of the WLA relied on the movement of the anchor after it was initially fastened to the stud and sheathing. The wire tie was mounted on 12 mm gypsum board for these tests. The lower bound adjustment was set at 10 mm from the wire leg to the centre of the fastening screw head. The upper bound of the adjustment range, at the farthest possible location from the wire leg, was found to be 30 mm.

From Figure 21.7d it can be seen that the behaviour at the lower bound was considerably stiffer than at the upper bound of the adjustment range. In the test at the upper bound, the eccentric loading caused bending of the wire and enough prying action on the screw to induce a loss of system

## TABLE 21.4 SUMMARY OF INFLUENCE OF RANGE OF ADJUSTABILITY

| TIE | TEST* | ULT. LOAD (N) | COV. (%) | LOAD @ 1.2 mm (N) |
|---|---|---|---|---|
| WAT | T-7-2[1] | 2641.8 | 4.6 | 829 |
| | T-7-4[2] | 2713.4 | 3.4 | 1061 |
| | T-7-4[3] | 3450.6 | 2.4 | 1277 |
| CTA | T-2-2[1] | 2139.2 | 1.8 | 105 |
| | T-2-4[2] | 1353.4 | 3.7 | 225 |
| | T-2-4[3] | 1742.1 | 5.2 | 254 |
| DW10 | T-9-2[1] | 2007.9 | 3.4 | 328 |
| | T-9-4[2] | 1966.1 | 5.3 | 265 |
| | T-9-4[3] | 2366.0 | 7.8 | 672 |
| DLA | T-1-2[4] | 1708.9 | 10.9 | 436 |
| | T-1-4[5] | 1139.8 | 10.4 | 236 |
| | T-1-4[6] | 572.0 | 2.5 | 78 |
| FLA | T-6-2[4] | 1128.7 | 6.1 | 506 |
| | T-6-4[5] | | | 206 |
| | T-6-4[6] | | | 53 |
| SSA | T-3-2[5] | 1167.5 | 3.6 | 582 |
| | T-3-4[7] | 1115.4 | 5.2 | 474 |
| | T-3-4[8] | 1000.4 | 4.1 | 331 |
| WLA | T-5-2[5] | 1287.5 | 4.6 | 484 |
| | T-5-4[7] | 1204.5 | 4.5 | 500 |
| | T-5-4[9] | 1101.2 | 6.4 | 283 |

Footnotes refer to adjustment location where:
1 = middle, 2 = quarter point, 3 = end
4 = minimum, 5 = 15 mm, 6 = 35 mm,
7 = 25 mm, 8 = 10 mm, 9 = 30 mm, and 10 = 20 mm.

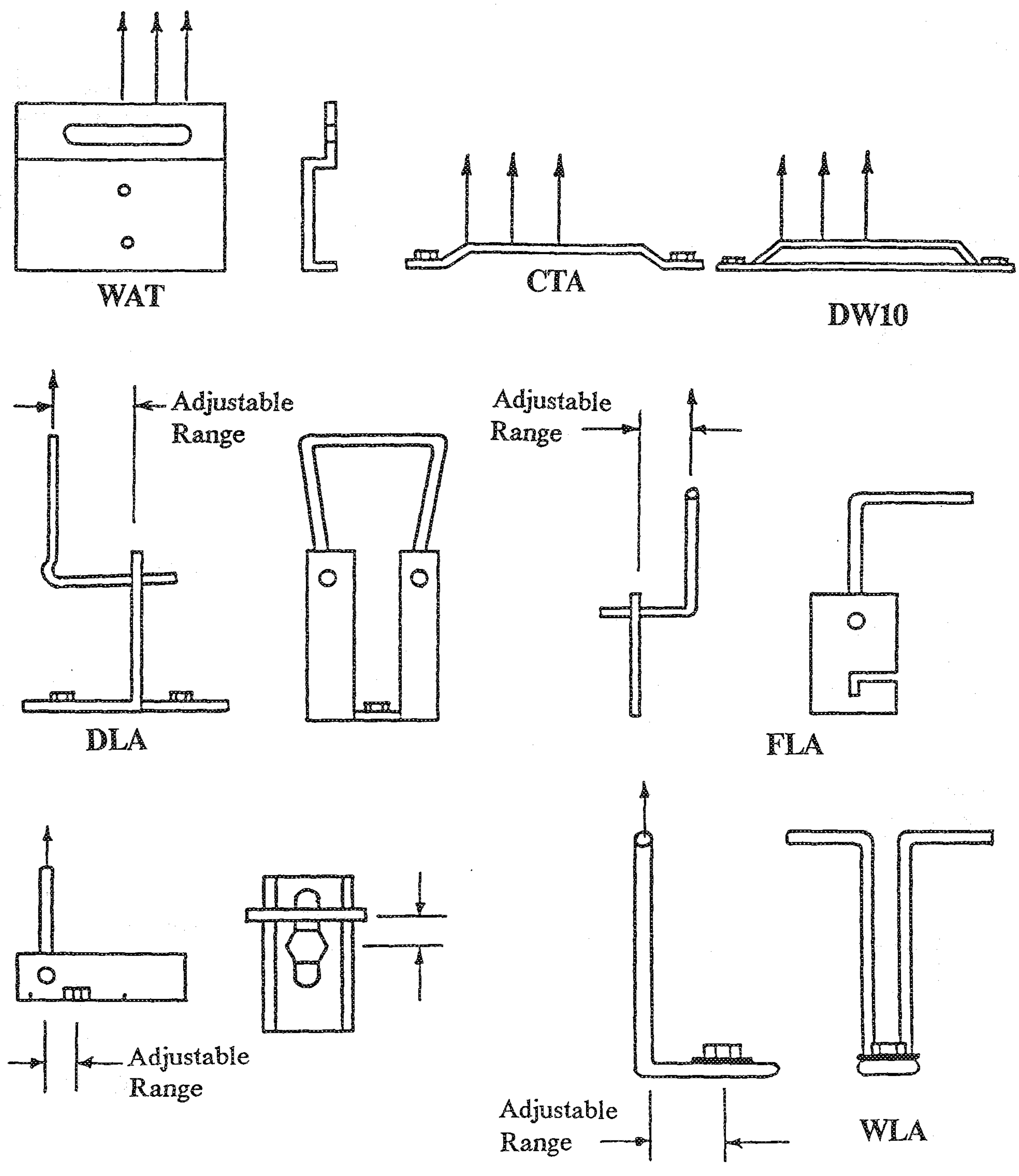


FIGURE 21.6 TEST CONFIGURATIONS FOR ADJUSTABLE RANGE

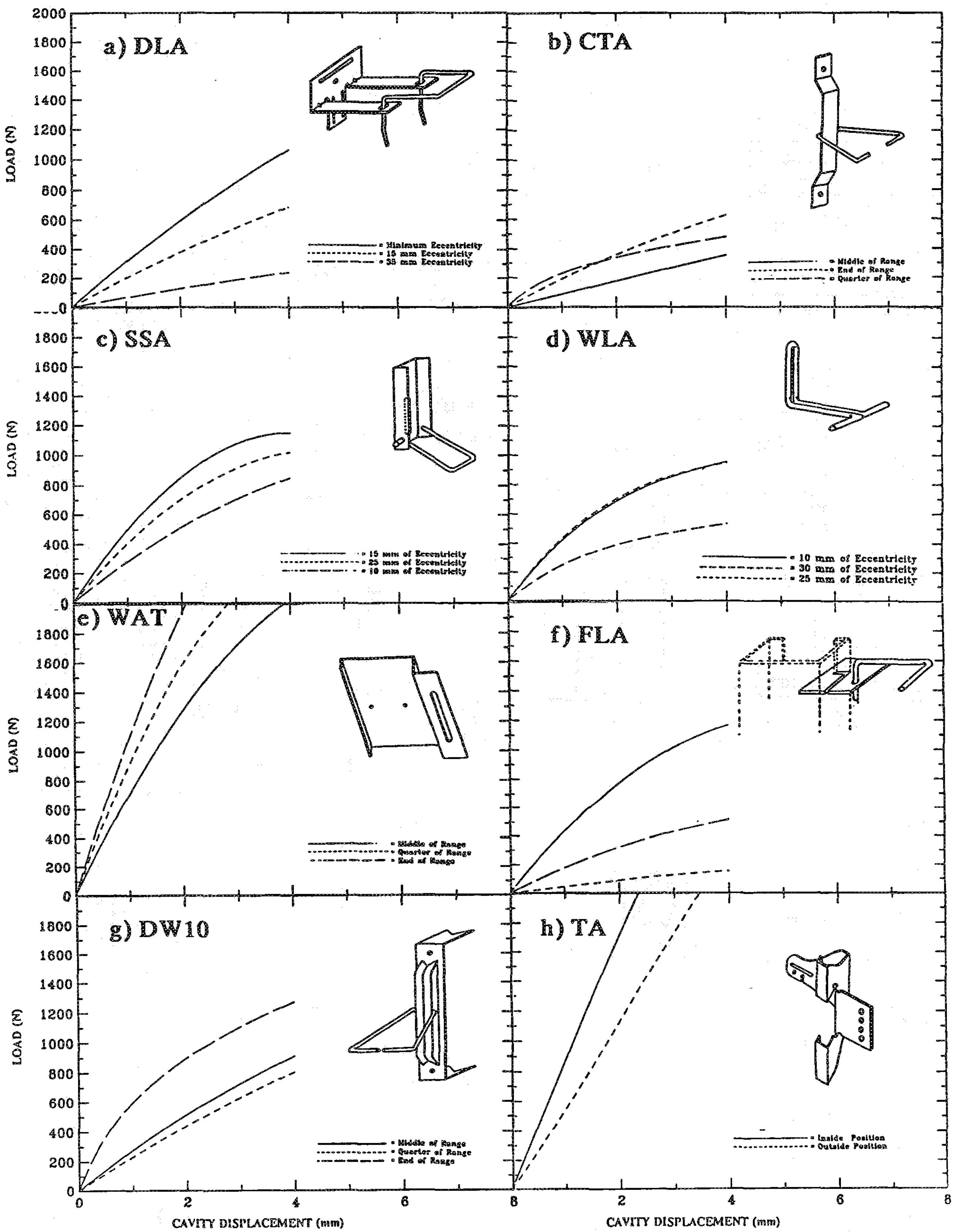


**FIGURE 21.7 INFLUENCE OF ADJUSTABLE RANGE ON PERFORMANCE OF VARIOUS TIES**

stiffness and reduce the tie's ultimate capacity. To further investigate this behaviour, an intermediate position in the range was investigated without applying the gypsum board. The results of the test were practically identical to the test at the lower range of adjustment with gypsum board, indicating that the gypsum board reduced the system stiffness.

* **T-6: Flange Locking Tie (FLA)**

The adjustment in the FLA was very similar to that provided in the DLA where the wire pintle was free to slide through the anchor and accommodate differences between the anchor line and the brick coursing. The resulting eccentric loading caused bending of the wire pintle which, as was shown in Figure 21.7e, resulted in decreased strength and stiffness with increased adjustment (eccentricity). The lower bound of the adjustment range with close contact between the anchor and the bend in the wire pintle corresponded to a minimum eccentricity of 5 mm. At the upper bound of the range, the anchor contacted the vertical pintle at a distance of 35 mm from the horizontal leg of the wire tie.

* **T-7: Wrap Around Tie (WAT)**

The WAT allowed the triangular wire tie to move through a vertical slot that extended into the cavity. The adjustable range of this tie was tested at the middle, quarter and end points of this slot.

The test results illustrated in Figure 21.7f showed that when positioned at the end of the slot, the tie possessed the greatest stiffness. It was most flexible when positioned at the middle of the slot.

* **T-9: The DW-10X Tie (DW10)**

The adjustment and behaviour of this tie was very similar to the CTA except that this tie did not fail with screw pull-out. The adjustment range was tested at the middle, quarter and end points.

When tested it was observed that the anchor itself yielded and deformed. This behaviour was most pronounced when the tie was at the quarter point of the anchor. The tie system was found to be stiffest when loaded at the end of the adjustment slot and nearest to a fastening screw. The results were shown in Figure 21.7g.

* **T-12: Thermosteel Tie (TA)**

This tie required a special type of steel stud to accommodate fastening. It provided four alternate holes to locate a triangular wire tie. These holes were located on a vertical strip that extended into the cavity space as was illustrated in Figure 21.4k.

Because of the different form of adjustability, these results were not included in Table 21.4. The test results shown in Figure 21.7h indicated that the specimens tested using the inside hole had greater stiffness than those tests using the outside hole. In both cases the ultimate failure resulted in the tie becoming disengaged from the thermosteel stud.

### 21.5.5 INFLUENCE OF GAUGE OF STEEL STUD ON TIE PERFORMANCE

In BV/SS construction, there often arises the situation where 18 gauge steel stud is used. Whereas the majority of tie tests performed using 20 gauge steel studs provided a lower bound, a small study was conducted to assess the influence of the gauge of steel on wall tie performance.

The additional tests were identical to the tension test on 20 gauge steel stud (Test Series T-?-2) and corresponded to Test Series T-?-6 of the test program. Table 21.5 contains the results of both series for the ties used. They were selected to represent a fair range of ties from compact stiff units to screw fastened and mechanically fastened ties.

For the CTA (Tie 2) and FLA (Tie 6) ties in which cases stiffness was limited mainly by the tie itself, there was only a small influence within the 4 mm displacement performance domain. However, beyond this domain the ultimate capacity of the CTA which was limited by screw failure, did increase as a result of using the 18 gauge stud. In fact, during the 20 gauge tests the screw simply pulled from the stud whereas for the 18 gauge tests it actually ripped through the stud material and in two cases the screw sheared off.

On the other hand, the SSA (Tie 3) showed a marked increase in stiffness when tested on the 18 gauge stud. In the SSA tests with 20 gauge studs, flange rotation contributed more significantly to the cavity displacement than was the case for the CTA or FLA tie tests. Taking this into consideration, it is reasonable to assume that the 18 gauge support reduced the flange rotation and served to stiffen the tie system. The mean load-deflection curves for these tests were plotted in Figure

TABLE 21.5 SUMMARY OF INFLUENCE OF STUD GAUGE

| TIE | TEST* | ULT. LOAD (N) | COV. (%) | LOAD @ 1.2 mm (N) |
|---|---|---|---|---|
| CTA | T-2-2[1] | 1708.9 | 10.9 | 104 |
| | T-2-6[2] | 2614.9 | 1.7 | 142 |
| SSA | T-3-2[1] | 1167.5 | 3.6 | 582 |
| | T-3-6[2] | 1883.1 | 2.5 | 843 |
| FLA | T-6-2[1] | 1128.7 | 6.1 | 506 |
| | T-6-6[2] | 1493.4 | 5.9 | 539 |

1. 20 gauge steel stud.
2. 18 gauge steel stud.

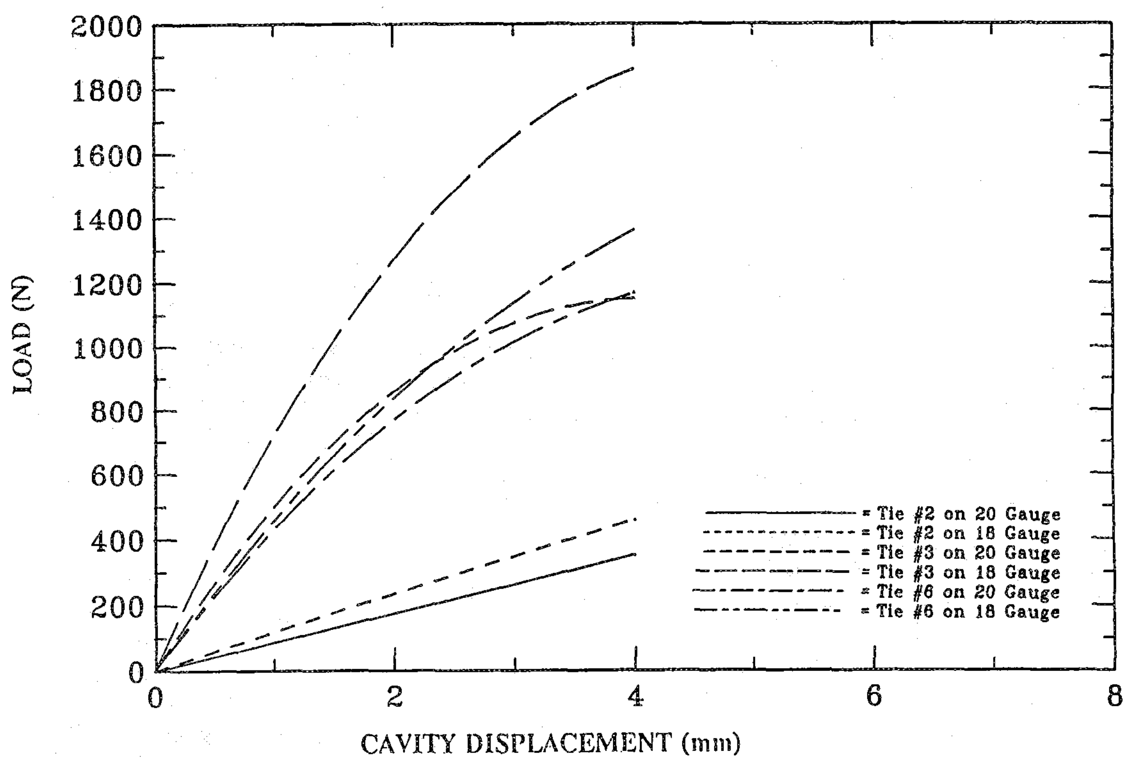


FIGURE 21.8 INFLUENCE OF GAUGE OF STEEL STUD ON TIE PERFORMANCE

21.8. Thus it can be observed that the influence of the gauge of the steel stud was very much dependent on the characteristics of the tie.

## 21.6 DISCUSSION OF REQUIREMENTS FOR BV/SS TIES

### 21.6.1 General Observations

One of the most obvious findings from this research is that all types of ties are not equal. In large part, this wide range of strength and stiffness can be attributed to the lack of established standards for design of ties. As a result, manufacturers have used criteria for design of BV/SS tie systems which differ markedly in the factors which seem dominant.

### 21.6.2 Requirements for Strength

Historically, the concept of a tie transferring load from a tributary area on the veneer (defined by horizontal and vertical tie spacings) to the backup wall has led to building code requirements specifying uniform maximum tie spacings for a few defined cases. Such a concept does lead to the conclusion that high tie loads are not encountered and that there is little difficulty in satisfying the strength requirements. However, even for a rigid backup wall such as reinforced concrete or concrete block, tie loads are far from uniform with the top tie resisting around half of the portion of lateral load carried by the veneer in bending. For flexible backup walls such as steel stud construction, the non-uniform distribution of tie loads is much more pronounced and creates a much more critical situation because the stiffer brick veneer resists the largest part of the lateral load regardless of whether the wind pressure is on the veneer or the backup wall[13]. If the brick veneer is cracked, then the very high top tie load is reduced but the tie nearest the crack becomes similarly highly loaded.

As a result of the above, it should be apparent that BV/SS ties must be designed to transfer loads which are much larger than those associated with the tributary area around individual ties. The magnitude of the load to be transferred will depend on the wind pressure related to the geographic location, exposure conditions and location on the face of the building, as well as to the span of the veneer and the relative stiffness between the brick veneer and the steel stud backup. This means that, unless it is considered to be satisfactory to have relatively conservative designs for most cases in order to satisfy extreme loading situations, no simple capacity requirement can be stated. This topic will be discussed further in Section 21.8.

### 21.6.3 Requirements for Stiffness

Large differences in strength were noted, not just between different ties but also for different sizes, cavity widths, locations of adjustment, attachment locations and sheathing materials for individual ties. Similarly large differences in stiffness were documented and study of the graphical presentations of this data clearly shows that some strengths are relatively meaningless where they correspond to very large displacements. Therefore it seems sensible to try to define a "Performance Domain" within which to judge the characteristics of ties. In this research, a performance domain of 4 mm displacement was arbitrarily selected but was considered to be a sensible limit.

Computer analyses[7,13] of wall systems have shown that, within reason, less stiff ties result in slightly lower tie loads and bending stresses in the veneer. An explanation is that the softer ties act "somewhat like shock absorbers" and result in a slightly more uniform distribution of the tie loads. However a certain minimum stiffness is required to ensure that movement of the veneer does not result in other problems such as fatiguing of components including non structural items such as caulking. In this regard, the performance recommendation of BIA[4] that ties develop 100lb. resistance at a displacement of less than 0.05 in. (444 N @ 1.2 mm) seems a reasonable compromise and a good starting point.

An added consideration for limiting the deformation of the tie under load is related to control of crack size. When veneer deflection is significantly more than the steel stud deflection as a result of tie deformation, the width of the crack in the veneer will be proportionally larger.

### 21.6.4 Construction Requirements

In the past, probably the single most dominant factor in choice of ties for construction has been economy both in terms of cost of the tie and in terms of time and convenience of installation which also has a cost value. The development of two component ties addresses convenience of use where interference of the ties with construction of the veneer is eliminated. Where adjustability is also incorporated, there is less dependence on careful positioning of the portion attached to the backup. Therefore these factors have been important in the development of BV/SS tie systems.

As a result of increased interest and discussion by the design professions, building officials and standards committees, other factors have begun to be recognized as equally important. Besides possessing some yet undefined minimum level of strength and stiffness, interactions of the ties with the building envelope have been identified. Where exterior sheathing is used, the provisions for placing the sheathing after the ties are fastened to the steel studs must be practical. Alternately, ties which are attached after the sheathing is in place must be designed to account for the mounting mechanism. In both cases the interactions of these ties with air and vapour barriers must be considered.

### 21.6.5 Other Considerations

Galvanizing of ties to provide corrosion protection is a standard requirement. However, it must be recognized that galvanizing is a sacrificial form of protection and, in corrosive environments, the protection will eventually be used up. Therefore, particularly for buildings which are expected to remain in use for long periods of time, use of stainless steel or other materials or methods of protection have become more common.

A consideration for tie design not previously dealt with in the subject of vulnurability. This can be assessed from many different aspects:

- robustness of the tie to stand up to field handling
- sensitivity to installation practices (i.e. overturning of screws, location of attachment and misalignment)
- potential for and consequences of failure (i.e. corrosion of screw hole, disingagement of tie)
- redundancy or otherwise ability to retain load carrying properties under damaged or overloaded conditions.

In view of the above, it can be appreciated that the increased awareness of the importance of ties to the long term performance of BV/SS walls has been and is leading to a more thorough consideration of design requirements.

## 21.7 DISCUSSION OF THE TEST RESULTS

In this section the discussion is focussed on general trends and on comparisons with performance recommendations for wall ties. Although no specific Canadian building code provisions exist to cover load capacity or minimum stiffness requirement for BV/SS ties, there have been some proposals[5,28]. Performance recommendations from the Brick Institute of America[5] specify that the tie be able to resist a load of 444 N (100 pounds) at a displacement of 1.2 mm. Whether the 1.2 mm of displacement should include the mechanical play inherent in most adjustable tie systems is questionable. However, such a restriction is not practical at this time since typical values for mechanical play for many of the currently available ties are 0.8 to 1.2 mm.

The tests were conducted so that the displacements did not include mechanical play. In part, this procedure was followed because anticipated future stiffness requirements may result in much reduced magnitudes of mechanical play.

Where applicable, the mean ultimate capacity of each tie was listed in the tables in Chapter 4. However the use of this single parameter to compare ties would be a mistake. For instance, although the CTA tie is one of the stronger ties, a closer examination reveals that its performance is well below BIA's definition of adequate stiffness. The regression equation coeffiecients were provided to facilitate further comparison at various limits of allowable displacement. In addition, the stiffness values obtained by dividing the loads at 1.2 mm displacement by 1.2 mm is an meaningful basis for comparison of tie properties in the service range of loading. In this way, some absolute requirement for stiffness may be evaluated.

From review of the test data, it is clear that no tie has a unique load-displacement relationship. In fact many factors can cause dramatic changes in behaviour. Probably, the most significant of these factors is the influence of adjustability of the tie. From Figure 21.9, which contains results for the ties specifically designed to incorporate adjustability, the range of behaviour is apparent. This observation applies both to the range of adjustability for a tie and to the differences between ties.

As an aid to comparing the performance of the previously defined classes of adjustable ties, a performance index (PI) has been defined. This index is an indicator of sensitivity of tie stiffness to use within its adjustable range. In interpreting the index, a value of 1.0 represents a tie for which stiffness is independent of adjustment, whereas lower values indicate increased sensitivity to adjustment. The index is calculated as :

$$PI = 1 - \frac{\text{(maximum stiffness - minimum stiffness)}}{\text{maximum stiffness}}$$

As an arbitrary but reasonable basis for comparison, mean PI values were calculated over the 0 to 4 mm displacement range.

The PI bar chart shown in Figure 21.10 permits comparisons between these 7 ties and between the classes of ties classified in terms of mechanism for adjustability. It can be observed that the CLASS 2 ties (relying on a wire pintle) performed very poorly compared to the others.

It is important to note that adjustability alone is not the only means of measuring a tie's performance. For instance, a comprehensive method of indexing BV/SS wall ties should also address the method and location of attachment. In this case, an attachment index (AI), rating of the CLASS 3 ties, (attached over the exterior sheathing) would likely be lowest as a result of increased probability of poor attachment and difficulty with accurate positioning on the stud flange.

Other factors such as misalignment, potential for improper installation (ie. incorrect bend point in a corrugated strip tie) and site damage could be included in a workmanship index (WI) rating.

Whether or not some formal rating system can be developed to aid in setting standards for design of ties, it is apparent that standards are needed. The next section deals with recommendations derived from the observations and conclusions of this BV/SS tie test program.

## 21.8 CONCLUSIONS AND RECOMMENDATIONS

### 21.8.1 Conclusions

Tie strengths and stiffnesses can be significantly affected by:

- positioning within the adjustable range,
- location of attachment on the steel studs,
- type of sheathing where ties are not directly attached to the studs,
- cavity width,
- gauge of the steel stud,
- misalignment or improper installation,
- and, although not included as part of this study, long term deterioration.

At the extremes of their adjustability, several of the tested ties have very low capacities and would not be adequate to resist the wind forces generated at top ties or at ties near veneer crack locations. Also many of the configurations tested did not satisfy the suggested maximum 1.2 mm displacement at a load of 450 N. As a result, it is recommended that prior to specifying a wall tie, test data be reviewed to ensure that it can perform its intended function at all possible degrees of adjustment and attachment locations. This requirement in itself would serve to place controls on the amount of adjustment possible for some ties. At present Can3-A370-M84[8] does not provide sufficient guidance for either tie manufacturers or designers to evaluate ties in a consistent manner. Therefore there is an identified need for this connector standard to be updated to include BV/SS type ties. However, consideration of tie requirements must be co-ordinated with other building code work

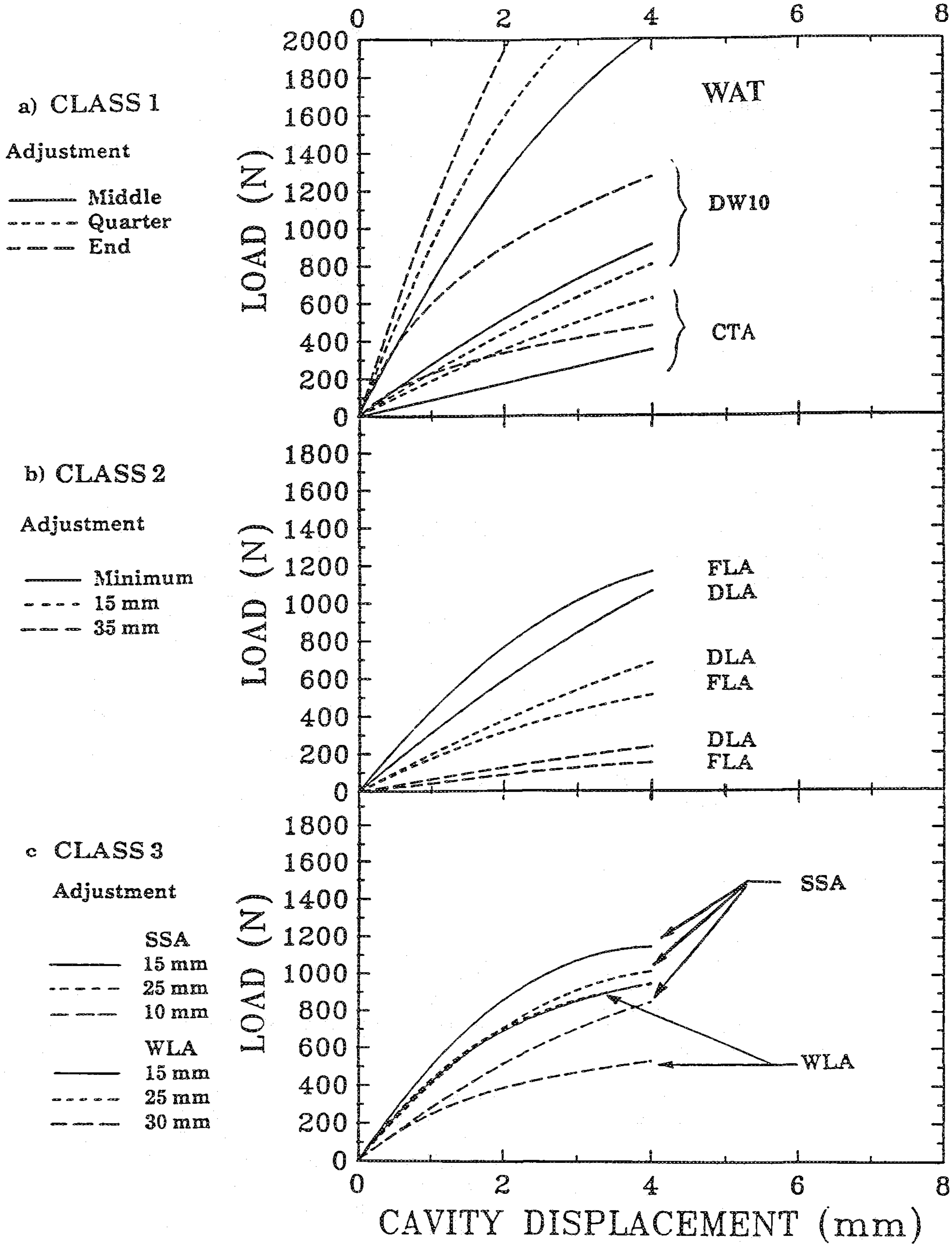


FIGURE 21.9 TENSILE TEST RESULTS FOR TIES WITH ADJUSTABLE RANGE

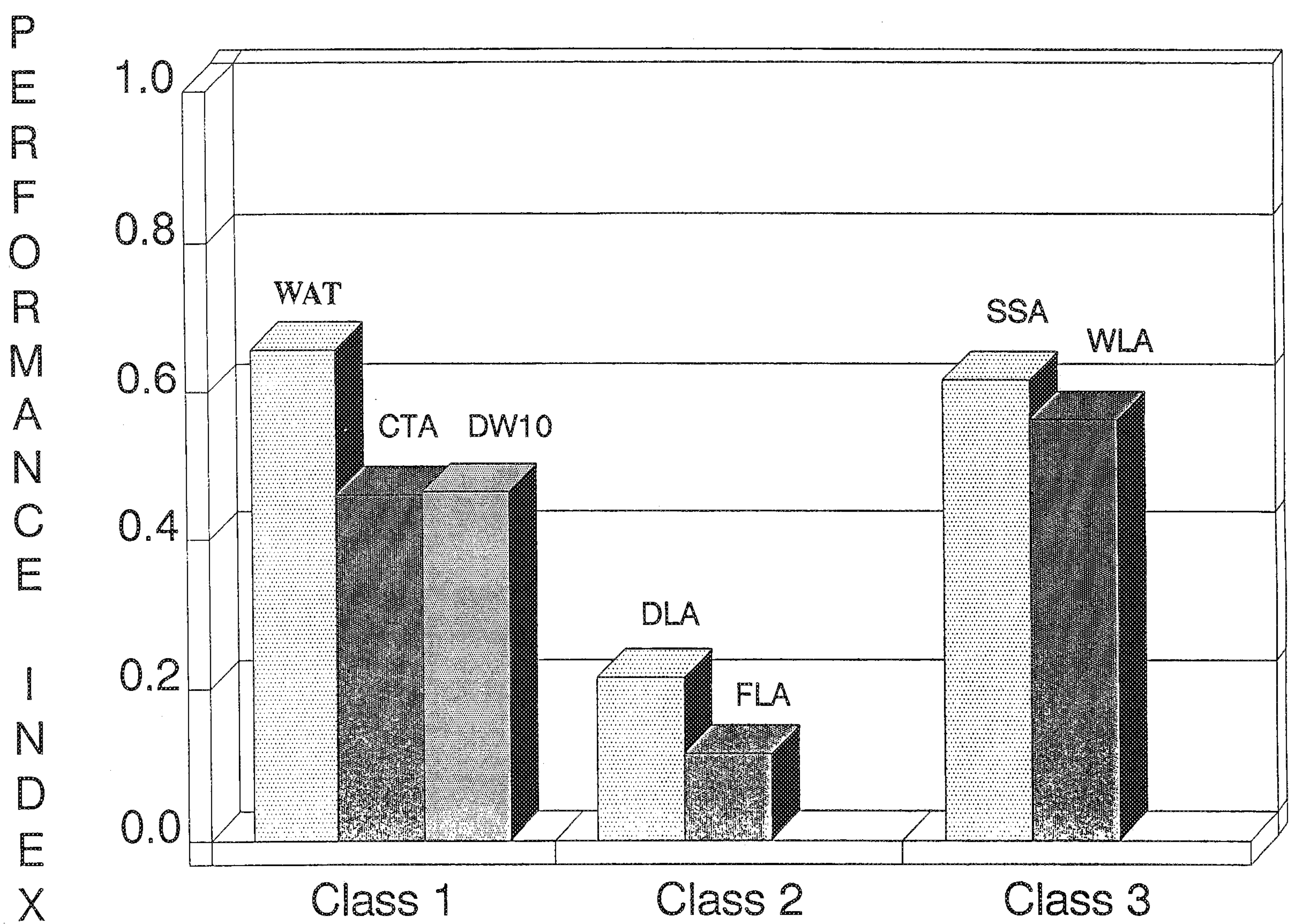


FIGURE 21.10 PERFORMANCE INDEX FOR ADJUSTABLE TIES

to define overall strength and stiffness requirements for the BV/SS wall system. In addition, due consideration must also be given to developing appropriate construction standards.

### 21.8.2 Recommendations

The general overall recommendation arising from this research is that BV/SS ties must be designed for the loads and conditions under which they are intended to be used. This requires that designers have access to reliable information on strength and stiffness. Where recommendations for arbitrary spacings and tie stiffnesses and strengths are provided as a design aid, these must be based on a comprehensive analysis of the influences of all the previously mentioned variables for the range of application of these aids.

In order that reliable and consistent information can be made available to code writing bodies and designers, it is necessary that standard test and reporting procedures be followed. It is suggested that the procedures shown in this report can serve as a guide to development of such a standard. In this respect it is very important that ties be evaluated as part of a system including steel stud, sheathing material, and screws or other attachment mechanisms. The characteristics of the tie should then be provided for the full range of adjustment, attachment location and misalignment.

## 21.9 CLOSURE

This research was carried out with the complete co-operation and openness of the tie manufacturers mentioned in the acknowledgements. Information provided here which reveals large differences in behaviour characteristics for various types of ties is indicative of the general problem that has become associated with BV/SS construction. Construction simply preceeded adequate research and the development of accepted standards for design and construction. It is anticipated that the test results and comments provided in this chapter will help interested parties reach a consensus on how to correct the current unsatisfactory situation.

REFERENCES


1. ARUMALA, J. O. AND BROWN, R. H., "Performance Evaluation of Brick Veneer with Steel Stud Backup", Dept. of Civil Engineering, Clemson University, Clemson S.C., April 23, 1982.

2. BRICK INSTITUTE OF AMERICA, "Brick Veneer New Construction", BIA Technical Notes on Brick Construction, 28 Revised, 1986.

3. BRICK INSTITUTE OF AMERICA, "Brick Veneer Existing Construction", BIA Technical Notes on Brick Construction, 28A Revised, 1987.

4. BRICK INSTITUTE OF AMERICA, "Brick Veneer Steel Stud Wall Panels", BIA Technical Notes on Brick Construction, 28B Revised II, 1987.

5. BRICK INSTITUTE OF AMERICA, "Wall Ties For Brick Masonry", BIA Technical Notes on Brick Construction, 44 Revised, 1987.

6. BRITISH STANDARDS INSTITUTION, "BS 1243:1978 Metal Ties for Cavity Wall Construction", British Standard, London, BSI, 1978.

7. BROOKS, M. S., "Masonry Veneer With Steel Stud Backup", M. Eng Project Report, McMaster University, Hamilton, Ontario, April 1985.

8. CANADIAN STANDARDS ASSOCIATION, "CAN3-A370-M84 Connectors for Masonry", Rexdale, Ontario, March 1984.

9. J.W. COWIE ENGINEERING LIMITED, "Investigation of Exterior Walls, Windows and Roof Alderney Manor, Alderney Drive, Dartmouth, Nova Scotia", Report for Central Mortgage and Housing Corporation, Halifax Office, 1976.

10. J. W. COWIE ENGINEERING LIMITED, "Laboratory Testing of the Cowie Brick Veneer/Metal Stud Wall System Used in the Restoration of Alderney Manor, Darmouth, Nova Scotia", 1985.

11. de VEKEY, R. C., "Corrosion of Steel Wall Ties:Recognition, Assessment and Appropriate Action", Building Research Establishment Information Paper IP28/79, Garston, BRE, 1979.

12. de VEKEY, R. C., "Performance Specifications for Wall Ties", Building Research Establishment Information Paper IP4/84, Garston BRE, 1984.

13. Drysdale, R. G., and Chidiac, S., "Defining Better Wall Systems", A Report submitted to the Research Division, Canada Mortgage and Housing Corporation, Ottawa, May 1989.

14. DUNCAN, R. L., "Brick Veneer/Steel Stud Wall Systems: An Overview", Proceeding Fourth Conference on Building Science and Technology, Toronto, Ontario. 1988.

15. GRIMM, C. T., "Metal Ties and Anchors for Brick Walls", Journal of The Structural Division ASCE, VOL. 102, No. ST4, April, 1976.

16. GRIMM, C. T., "Distressed Exterior Masonry Walls", Structural Report, 1984.

17. HASELTINE, B. A., "Report on Causes of Failures and Methods of Restoration of The Exterior Walls of Alderney Manor, Darmouth, Nova Scotia", Report for Kitz Matheson Barristers and Solicitors, Halifax, Nova Scotia, 1984.

18. HATZINIKOLAS, LEE, LONGWORTH AND WARWARUK, "The Behaviour of Styrofoam SM Insulation As Exterior Sheathing In Brick Veneer And Metal Stud Wall System", Department of Civil Engineering, University of Alberta, Edmonton, Alberta, 1986.

19. HATZINIKOLAS, M. A., LEE, R. AND WARWARUK, J., "Factors Affecting the Performance of Metal-Stud Masonry Veneer Wall System", Proceedings Four Conference on Building, Science and Technology, Toronto, Ontario, 1988.

20. HATZINIKOLAS, M. A., LONGWORTH, J., AND WARWARUK, J., "Brick Veneer Tie Support", Testing Report, (prior 1984).

21. HEIDERSBACH, R., BORGARD, B., AND SOMAYAJI, S., "Corrosion of Metal Components in Masonry Buildings, (post 1986).

22. HODGKINSON, H. R., "Structural Tests on Aspects of Clay Brick Masonry Veneer With Steel Stud Backup Wall System for J. W. Cowie Engineering Limited", Structural Testing Report by British Ceramic Research Association Limited, June 1985.

23. KELLER, H., "Brick Veneer/Steel Stud Wall Design and Construction Practices in Canada", Results of a 1986 Survey Prepared by Suter Keller In Project Implementation Division of Canada Mortgage and Housing Corporation, 1986.

24. MCGINLEY, W. M., WARWARUK, J., LONGWORTH, J., AND HATZINIKOLAS, M., "Masonry Veneer and Steel Stud Curtain Walls", Publication of the Prairie Masonry Research Institute, 1986.

25. MOORE, J. F. A., "The Performance of Cavity Wall Ties", Building Research Establishment Current Paper CP3/81, Garston, BRE, 1981.

26. PLEWES, W. G., "Exterior Wall Construction in High Rise Buildings", Advisory Document, Canada Mortgage and Housing Corporation, 1981.

27. STANDARDS ASSOCIATION OF AUSTRALIA, "AS 2699-1984: Wall Ties For Masonry Construction", North Sydney, N. S. W., 1984.

28. ZAKRZEWSKI, A. S., "Masonry Veneer in Highrise Buildings", Report Submitted to Canada Mortgage and Housing Corporation by Proen Consultants, Willowdale, Ontario, 1987.

## Chapter 22

# A SUMMARY OF

# TESTS OF FULL SCALE BRICK VENEER / STEEL STUD WALLS TO DETERMINE STRENGTH AND RAIN PENETRATION CHARACTERISTICS

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RGD-97b

## 22.1 GENERAL

This report contains information on the design and construction of a research facility for testing brick veneer/steel stud (BV/SS) walls subject to lateral wind pressure. In addition to wind loading, the facility was equipped to perform rain penetration studies on the test walls. The research reported comprises Part 5 of a five-part laboratory test program at McMaster University sponsored by the Project Implementation Division of Canada Mortgage and Housing Corporation. While information in this report has some relevance to other brick veneer wall systems, this investigation was focussed solely on the performance of the combined BV/SS wall system.

### 22.1.2 Criteria for Design of Test Facility

Having decided to perform tests on full scale BV/SS walls, it was important to try to design a test facility with as much flexibility as possible in terms of the types of information which could be gathered. For general acceptance of the test results and to permit comparisons with other test results or with existing code requirements, it was desirable to follow recognized test procedures as closely as possible.

Using relevant tests described in ASTM standards, the test methods listed in Table 22.1 were reviewed and used as the basis for designing specific aspects of the wall test rig.

In all but ASTM E 514, the test methods can be applied to windows, curtain walls, or doors. As will be indicated in the following discussion, it was intended that the apparatus possess sufficient versatility to be extendable to a wide range of applications.

TABLE 22.1

STANDARD ASTM TEST METHODS APPLICABLE TO DESIGN OF WALL TEST RIG

| ASTM Reference Designation | Test |
|---|---|
| E 283 | Air Leakage |
| E 330 | Structural Performance |
| E 331 | Water Penetration |
| E 514 | Water Penetration (Masonry) |
| E 547 | Water Penetration (Cyclic) |
| E 1233 | Structural Performance (Cyclic) |

### 22.1.3 Structural Tests

For structural testing, provision to provide positive or negative, static or dynamic air pressure loads of up to 12.5 kN/m$^2$ (250 psf) was set as a design objective. Tests were intended to be able to provide the following information:

- absolute and relative deflections of the veneer and stud backup,
- effect of veneer cracking on deflections and on overall wall stiffness,

- cracking loads and ultimate strength of the BV/SS wall system, and
- effect of cyclic loading on wall stiffness.

With air pressures applied to either the veneer or the backup as either positive or negative pressure, the full range of load transfer mechanisms can be tested. For load on the veneer, the ties will act in compression or tension to transfer the load to the backup wall. Alternatively, as shown in Figure 22.1, when the cavity is pressurized, the external loads on the veneer are balanced and deflection of the backup will cause tie forces. Also shown is the case where the cavity is partially pressurized which will result in pressure drops across both the veneer and the backup wall.

To apply air pressure to either the veneer or the backing requires that both be sealed into the apparatus reasonably well so that the test wall forms one side of the pressure chamber.

### 22.1.4 Environmental Tests

With the versatility of cavity pressurization configurations shown in Figure 22.1, the performance of the veneer in terms of the rain screen principle can be thoroughly investigated. Also, because of the capability of introducing a crack in the veneer, this testing can be extended into the cracked veneer range.

Furthermore, the equipment is well suited to do tests to determine the air tightness of the backup wall and in particular the as-constructed quality of the air barrier.

## 22.2 CONSTRUCTION AND INSTRUMENTATION OF THE FULL SCALE WALL TEST APPARATUS

### 22.2.1 General

WALTER (Wall Test Rig) was designed to consist of two self-contained and independently stable components, namely the Specimen Frame (shown on the left in Figure 22.2) and the Pressure Chamber (shown on the right in Figure 22.2). As the name implies, the Specimen Frame was intended to support and house the wall specimen during testing. As shown in the drawing in Figure 22.3, it was designed to be supported by columns bolted to the laboratory strong floor. In this fixed position, construction of test walls can proceed with adequate clearance on all sides and the Pressure Chamber can be rolled into place when required.

For storage the Specimen Frame can be unbolted from the support columns and left attached to the Pressure Chamber. The option does exist to leave the Specimen Frame bolted to the Pressure Chamber, in which case support columns are not required. The Pressure Chamber was designed to be completely portable and if required can be used in conjuction with several Specimen Frames.

### 22.2.2 Air Pressure Supply

Air pressure in the chamber is provided by a 3550 RPM cast iron Buffalo Pressure Blower (Model 4RE) manufactured by Canadian Blower/Canada Pumps Ltd. The blower has a 650 mm diameter wheel and is rated for a maximum pressure of 11.5 kPa. The blower is directly driven by a three phase 480 volt DC, 3500 RPM motor manufactured by General Electric. The motor frame size designation is CD 218 AT and it is rated at 15 horsepower. The motor speed is controlled by a matching custom manufactured motor controller unit designed and built by Serv-e-tronic Ltd. The blower, motor and cabinet of the controller is shown in Figure 22.4a).

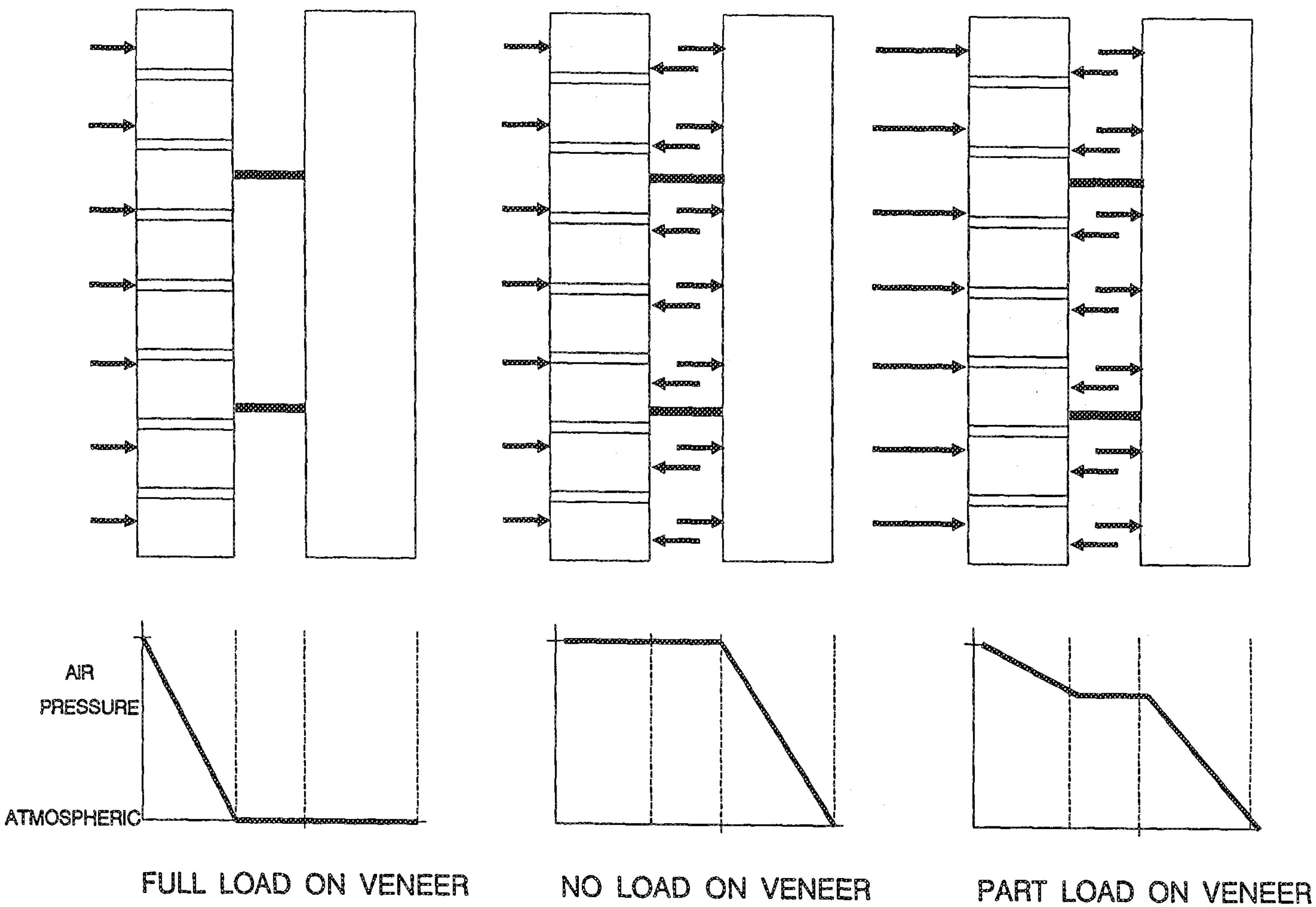



**FIGURE 22.1 ILLUSTRATION OF POSITIVE AIR PRESSURE CONFIGURATIONS**

FIGURE 22.2 PHOTOGRAPH OF THE TWO COMPONENT PARTS OF WALTER

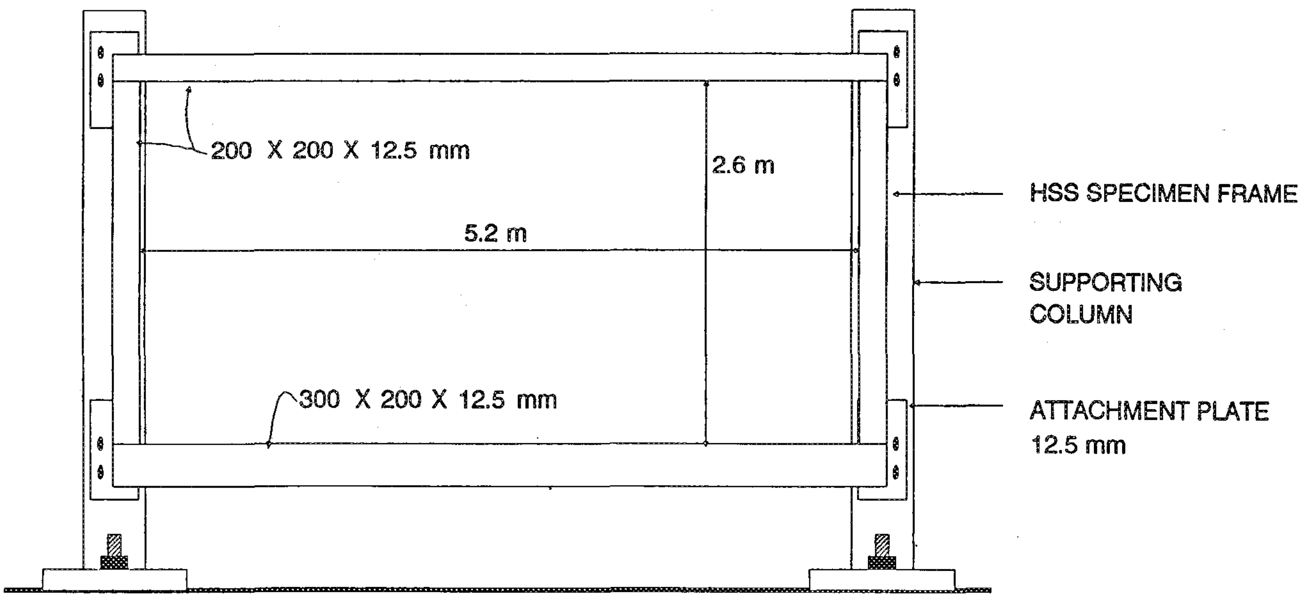



FIGURE 22.3 DRAWING OF SPECIMEN FRAME

The controller unit was built to operate as a stand-alone unit or as a slave to a computer control system. At present it is operated as a stand-alone unit . Small increments of motor speed are attainable along with highly precise speed control through signal digitization. The absence of reference signals allows the motor to be dynamically controlled in both the run mode and, for safety reasons, in the disabled mode.

### 22.2.3 Water Spray Equipment

The water spray equipment to simulate rain is visible in Figure 22.2 and is shown schematically in Figure 22.4b). A water spray over the surface of the wall is created by 28 full cone wide angle spray nozzles manufactured by Spraying Systems Co. The design data provided by the manufacturer was used to choose the nozzle (1/8-GG-8W), operating water pressure (68.9 kPa) and spraying distance (300 mm) to obtain a wall coverage of 5.6 L/min/m$^2$. The brass spray nozzles are attached via split eyelet connectors to the 19 mm looped copper tubing grid shown in Figure 22.4b). A 30 mm copper pipe is used to feed the grid from one end. The closed loop design ensures uniform pressure characteristics throughout the grid.

The system was designed to be recirculating. Water that collected in the bottom of the Pressure Chamber drains out and returns to a water reservoir tank externally attached to the chamber. A pump then draws water from this tank, passes it through a filter and then back into the water spray grid.

### 22.2.4 Equipment for Measurements

Structural: For the structural aspects of the tests, applied loads and the resulting deformations and forces in members are of primary interest.

Applied lateral load in the form of air pressure is measured with inclined and U-Tube manometers which are also used to set the required motor speed for the blower. Pressure transducers are used to measure and record air pressure data at various locations in the pressure chamber and in the wall test specimen.

Deflections of the steel stud backup wall and of the brick veneer are measured with mechanical dial gauges and LPDT's (Linear Potentiometric Displacement Transducers) attached to a mounting grid fastened to the Specimen Frame. Since deflection of the veneer is ideally measured from the exterior surface, and since access to the Pressure Chamber is impossible during a loading test, it is necessary to use transducers to measure veneer deflection.

Of considerable interest as well is the distribution of brick tie forces. This is measured indirectly using electric strain gauges carefully applied to selected ties. Conversion of the strains and direct calibration allows tie forces to be obtained.

An Optilog data acquisition system connected to a micro computer is used to record data from all electronic devices and for manual input of other data or test information.

Air Leakage: At present the air flow rate into the chamber is measured using a 4 mm modified ellipsoidal nose Pitot Static tube located at the centre of the 1.2 m length of straight ABS pipe. This length satisfies the recommended straight pipe length for pitot tube measurements.

Water Penetration of the Veneer: Water which penetrates the veneer and runs down to the bottom of the cavity is collected and weighed in order to obtain a quantitative measure of rain penetration. Also the change in volume of the water in the closed loop system is measured to determine how much water was absorbed by the wall.

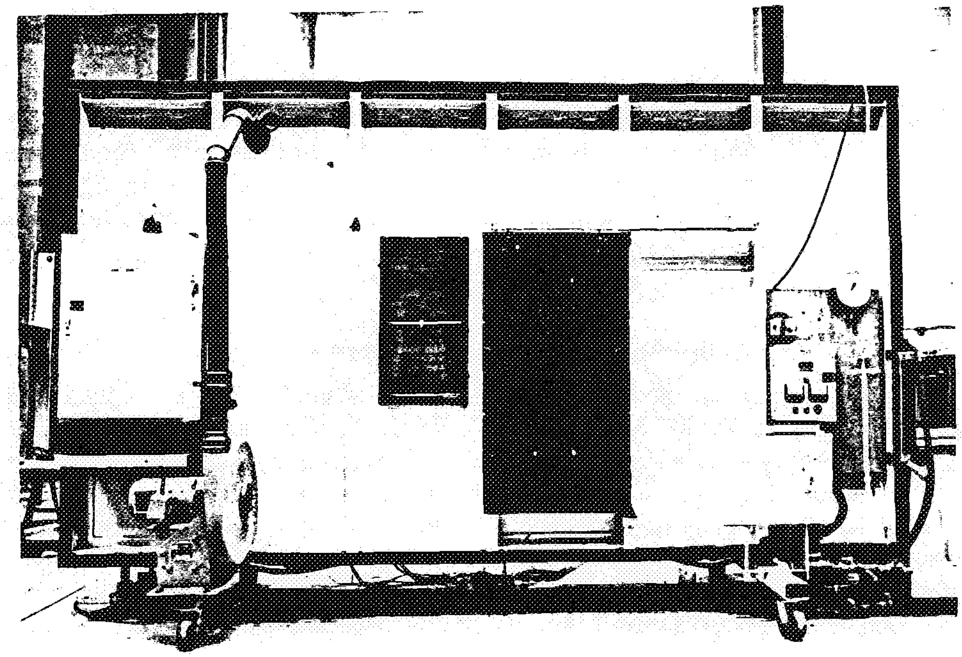


FIGURE 22.4a) EXTERIOIR VIEW OF PRESSURE CHAMBER ATTACHED TO SPECIMEN FRAME

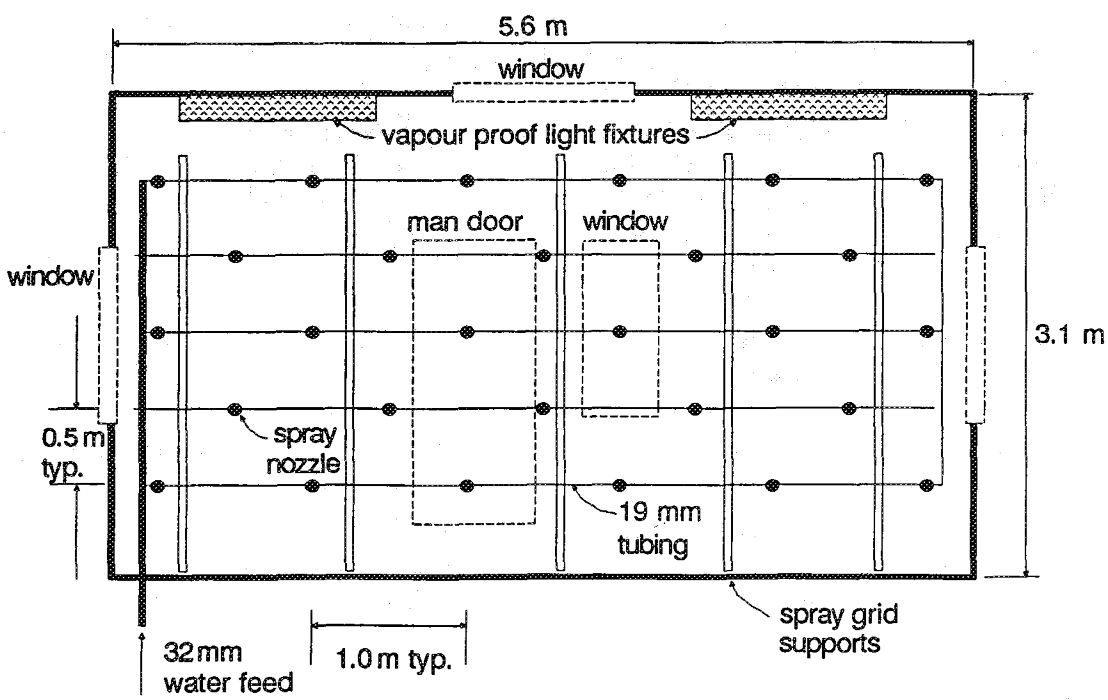



FIGURE 22.4b) INTERIOR VIEW OF PRESSURE CHAMBER INCLUDINGF WATER SPRAY GRID

## 22.3 DESCRIPTION OF WALL TEST SPECIMENS, TEST PROCEDURES AND PRESENTATION OF RESULTS

### 22.3.1 Introduction

The wall test specimens described in this chapter were used to investigate both the structural response and rain penetration characteristics of BV/SS wall systems.

At this stage, 2 nearly identical walls have been tested. The principal points of interest for these tests are briefly reviewed in the following 2 sections. In addition, a major objective of the early part of this test program involved verifying the capabilities of the test facility, developing test procedures and gaining experience with operating the equipment. Also, a not inconsequential aspect was the development of construction skills for fabrication of the test specimens.

For structural tests, the objectives were to document the behaviour characteristics of the brick veneer, the tie systems, and the steel stud backup wall. Of particular interest was the influence of pressure on the veneer versus pressure on the backup, and the behaviour before cracking of the veneer compared to after the veneer had cracked. In addition, cracking loads, locations of cracks, ultimate capacity and failure mechanisms were all important data to be gathered.

Observations of behaviours of stud to track connections, torsional bracing, tie connections and connection of sheathing were also of interest.

The principal features of the rain penetration tests were to provide information on:

- the permeability characteristics of brick veneer walls,
- the correlation between rain leakage and identifiable areas of imperfections or poor workmanship,
- the effectiveness of brick veneer as a rain screen,
- the effectiveness of the open rain screen concept as protection against rain penetration,
- the influence of partial pressurization of the cavity and the influence of air leakage through the backup wall,
- adequacy of vents to achieve pressure equalization for BV/SS walls designed on the open rain screen principle.

In all of the above, the influence of air pressure levels and, in particular, the effect of cracking of the brick veneer were important variables.

Because of the many factors to be investigated, it was necessary that the test program for each test wall be designed to include test sequences to provide information on as many aspects as possible.

### 22.3.2 Design and Construction of The Wall Test Specimens

- **Wall Test Specimen 1**

The construction of the first test wall was representative of normal building practice. Measuring 2.73 meters high by 5.2 meters long, the brick veneer panel had a surface area of 14.2 $m^2$. The steel stud backup wall was designed to a deflection limit of L/720 for a 1 $kN/m^2$ (20 psf) air pressure applied uniformly to the stud. (This naturally converts to a deflection limit of L/360 for a 2$kN/m^2$ (40 psf) air pressure.) To satisfy the above deflection limit, 18 gauge, 92 mm deep steel studs were required at 40 mm spacing (16 inch). This spacing of studs matched the gypsum board which came in 4 foot (1.22 m) widths.

In keeping with standard practice, one line of through-the-web bridging was placed at the mid height knock-out holes in the web. The channel shaped bridging was attached to the studs using an 18 gauge clip angle and double sets of screws.

The studs were framed at the bottom with 18 gauge low leg track and attached with screws in both flanges. A nested track arrangement with a 12 mm movement joint was constructed at the top of the wall and the stud was attached by screws on both flanges to the longer legged inner track.

The 12.5 mm thick exterior grade gypsum board was screwed to the studs with self-drilling drywall screws spaced at 204 mm vertically except at the edges where the spacing was 102 mm. Tar impregnated building paper was placed in overlapping sheets on the exterior of the exterior grade gypsum board after which the adjustable wire loop anchor type of tie was attached. These ties were mounted using a brick veneer tie support to provide direct contact through the gypsum board to the flange of the steel studs.

The wire loop anchors were attached through the tie support platforms and directly to the flanges of the studs using Teks 13 No. 10-16×1 HWH/2 self-drilling screws. The vertical spacing on each stud was 610 mm providing a maximum tributary area of 0.248 $m^2$. The top ties were located 203 mm from the top of the wall creating a nominal tributary area of 0.206 $m^2$. It should be noted that the ties were accurately placed so that very little adjustment of the ties was necessary.

The interior 12.5 mm thick gypsum board was attached to the studs with self-drilling dry wall screws spaced at 400 mm vertically on the steel stud except that this spacing was halved along the edges.

The steel studs adjacent to the sides of the Specimen Frame were not attached except in the normal manner at the top and bottom tracks.

Yellow clay bricks supplied by Canada Brick were of high quality and showed very little evidence of fire cracks. The standard 190 mm long by 57 mm high bricks resulted in 41 courses of 26 bricks per course allowing for the standard mortar joint. Type S mortar mixed using proportions of Portland Cement: Masonry Cement: Sand of 1:0.5:4.5 by volume provided a mortar satisfactory to the mason.

A 17 mm movement joint was left at the top of the wall between the brick veneer and the upper shelf angle. A 25 mm diameter foam rope was forced into the horizontal gap between the top brick and the shelf angle. Silicone sealant was then used to caulk the joint to prevent water penetration.

The Wire Loop Anchor tie mounted on a brick veneer support device was used for this wall. The tie and mount were chosen to ensure that tie failure would not initiate failure of the wall within the design range of loading.

- **Wall Test Specimen 2**

Wall Test Specimen 2 was constructed in a manner identical to Wall 1 except that a self-drilling tie was used. Also the steel studs adjacent to the side members of the Specimen Frame were attached to these H.S.S. sections at several points over their heights.

The construction of the Wall Test Specimens were carried out in the laboratory under full supervision conditions. While no special instruction was given, it is likely that the quality of construction was better than is normally achieved in practice. An experienced mason built the veneer and experienced personnel constructed the backup wall. Visitors to the laboratory commented on the apparent good quality of the construction.

Brick prisms constructed in a stack pattern were used to evaluate the flexural tensile bond characteristics of the combination of bricks and mortar used. They were made using mortar batches corresponding to construction of the mid-height portion of the veneer. A bond wrench was used to determine the mean flexural tensile strength of 0.72 MPa.

### 22.3.3 Structural Test Results

- **Wall Test Specimen 1**

Prior to building the veneer, the stud backup for Wall 1 was loaded to document its independent load-displacement characteristics. Figure 22.5 shows the deflection profiles for the centre stud for air pressures up to 1 kPa. Actual deflections were slightly less than the theoretical values because the gypsum board sheathing provides more end restraint then assumed in the calculations.

At the top the veneer deflected considerably more than the backup wall whereas the values were closer at mid height. At a pressure of around 1.4 kPa, non-linear behaviour was evident. This is interpreted as indicating that the veneer had begun to crack at some slightly lower pressure.

At loads over 4 kPa, extensive deformation was observed and as shown in Figure 22.6, eventually crushing of studs and collapse of ties was observed.

Figure 22.7 is a photographic view of the backer rod in the movement joint at the top of the brick veneer. The fact that it restrained local displacement is obvious and rough calculations revealed that for the way it was forced into the movement joint, this filler material would have a stiffness approximately equal to a line of ties at this level.

- **Wall Test Specimen 2**

Wall Test Specimen 2 differed from the first test in that the end studs were supported over their full height. This would tend to cause the wall to act as a plate supported on four sides. Figure 22.8 indicates that the stud deflections for load on the backup caused greater deflections of the middle stud than for a stud at the quarter point along the panel length. In this test, no material was placed in the movement joint at the top of the veneer.

### 22.3.4 Rain Penetration Tests

- **Wall Test Specimen 1**

At this stage, data from Wall 1 is not presented in detail. However some interesting general observations are provided. Principal among these observations are:

- Water leakage through the veneer was substantially larger when the cavity was not pressurized.
- During the initial test it was found that the caulking around the perimeter of the interior gypsum board was not an effective air barrier resulting in large air leakage rates. The consequence of this high leakage was that water was observed to "climb the track" in the backup wall and to "jet across" the cavity at a few other locations. This was a graphic illustration of the energy available to transport water across the cavity and through the backup wall.

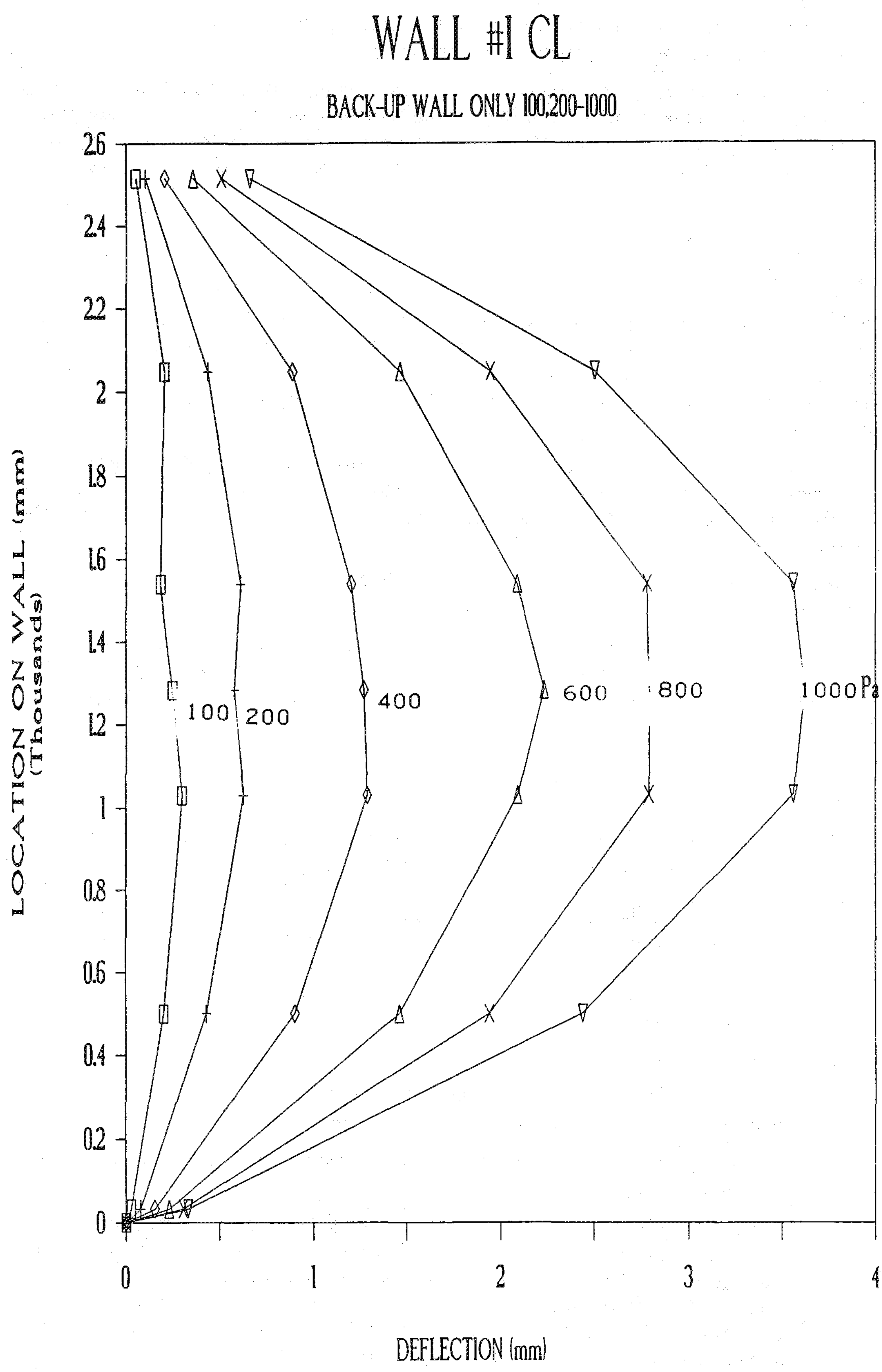



**FIGURE 22.5** **DEFLECTIONS FOR CENTRE STUD OF TEST WALL 1 LOADED BEFORE VENEER WAS BUILT**

**FIGURE 22.6**

**PHOTOGRAPH OF STUD FAILURE AT A TIE**

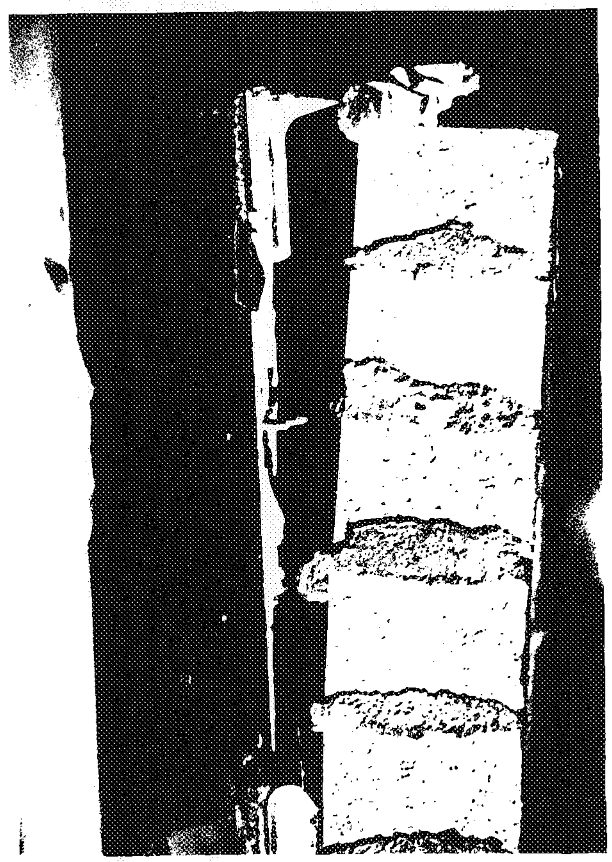


**FIGURE 22.7**

**PHOTOGRAPH OF BACKER ROD USED IN MOVEMENT JOINT**

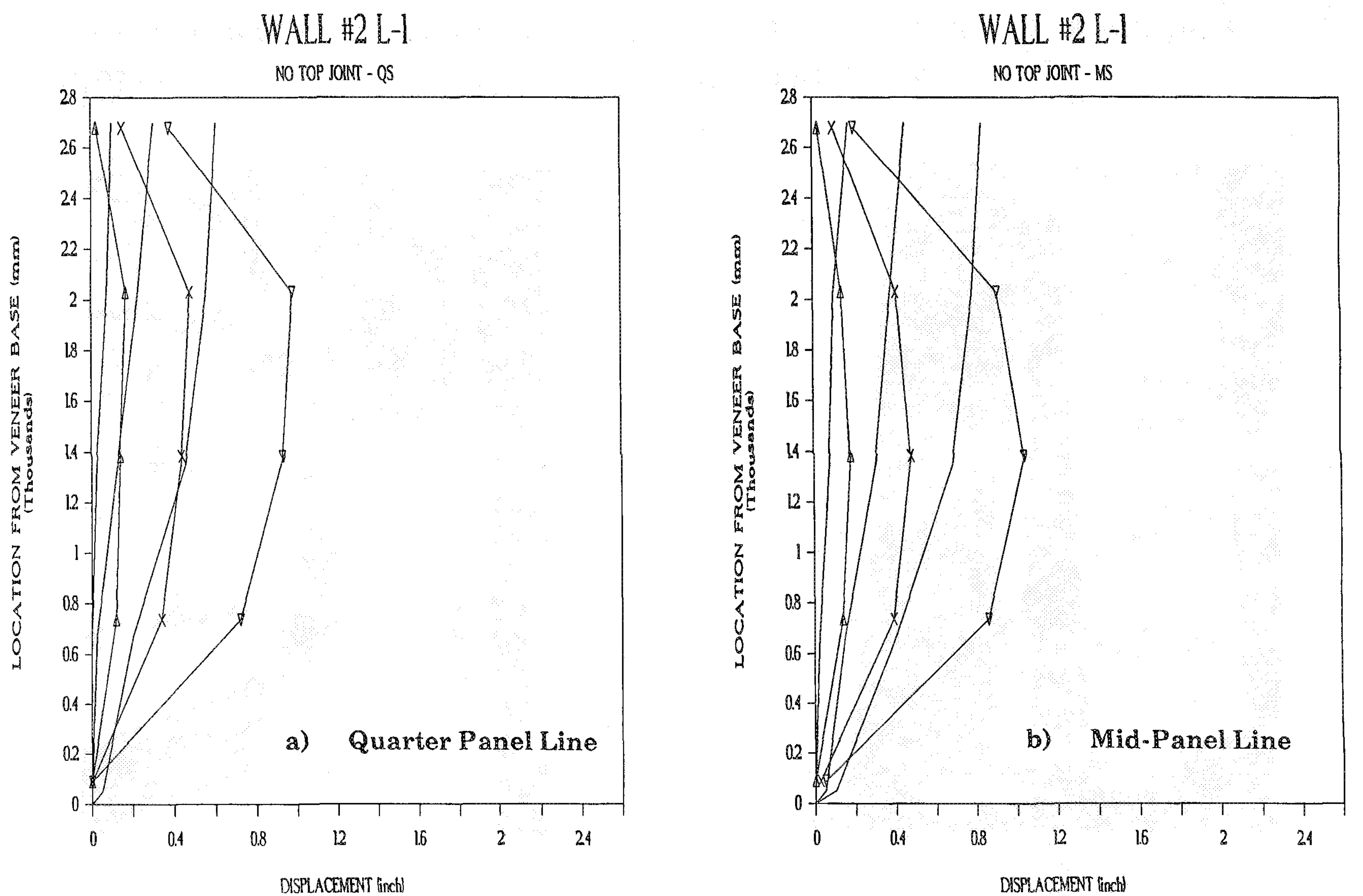



**FIGURE 22.8** DEFLECTIONS OF BACKUP AND VENEER FOR TEST WALL 2 WITH NO TOP OF VENEER SUPPORT

- Resealing the air barrier allowed testing to continue and demonstrate much larger magnitudes of rain penetration after the veneer had cracked compared to precracked leakage values.

- **Wall Test Specimen 2**

The results of the tests on Wall 2 indicate that the wall specimen possessed very uniform leakage characteristics without large individual leakage paths. This was determined by comparing results of leakage rates over different portions of the wall. Specifically it was found that the leakage rate per square meter of wall is fairly constant over different sections. Results for the three investigated parameters related to performance of the rain screen are discussed below:

To investigate the influence of construction on the wall, loadings 2 and 3 were conducted with a poorly sealed top joint. The poor top joint was produced by improperly caulking in front of the foam rope backing.

The results, presented graphically in Figure 22.9, show that the poor top joint contributed greatly to the overall leakage rate of the wall.

After the leakage rates for both the cavity vented and cavity equalized conditions of loadings 4 and 5 were determined, the wall was loaded in excess of 2 kPa to induce cracking. The cracked brick veneer wall panel was then subjected to rain loading again.

For the cavity equalized condition (active rain screen) the cracked veneer did not significantly influence the leakage rates. In comparison, for the cavity vented condition (non-active rain screen) whose leakage rates were greatly influenced by cracking of the veneer. This is the basic premise of the rain screen principle. That is, even though the veneer was cracked there was no force (pressure difference) to drive any contained water (capillary) into the cavity and therefore there was little increase in leakage for the active rain screen case until later in the test.

Another requirement of the rain screen principle is that the rigidly formed cavity be rapidly pressurized because of gust effects. The importance of this requirement was investigated through the influence of a pressure gradient across the rain screen. The longer that the equalization of the cavity is delayed, the greater the pressure difference across the rain screen. Certainly the time required is important to both the structural response and the performance of the rain screen. This facet of the wall system forms yet another aspect of the research which is not presented here.

The results shown illustrate how important it is to achieve pressurization of the cavity for either a cracked or uncracked veneer wall. Regardless of the cracked condition of the veneer, the leakage rate rose significantly with increases in pressure differences. It should be noted that the greater leakage rates for the cracked wall is attributed to increased leakage paths.

The time delay and the overall ability to achieve pressure equalization will depend on the following factors:

1. Air tightness of the backup wall.
2. Degree of compartmentalization of the cavity.
3. Size of vents in the brick veneer.
4. Volume of air to be compressed.

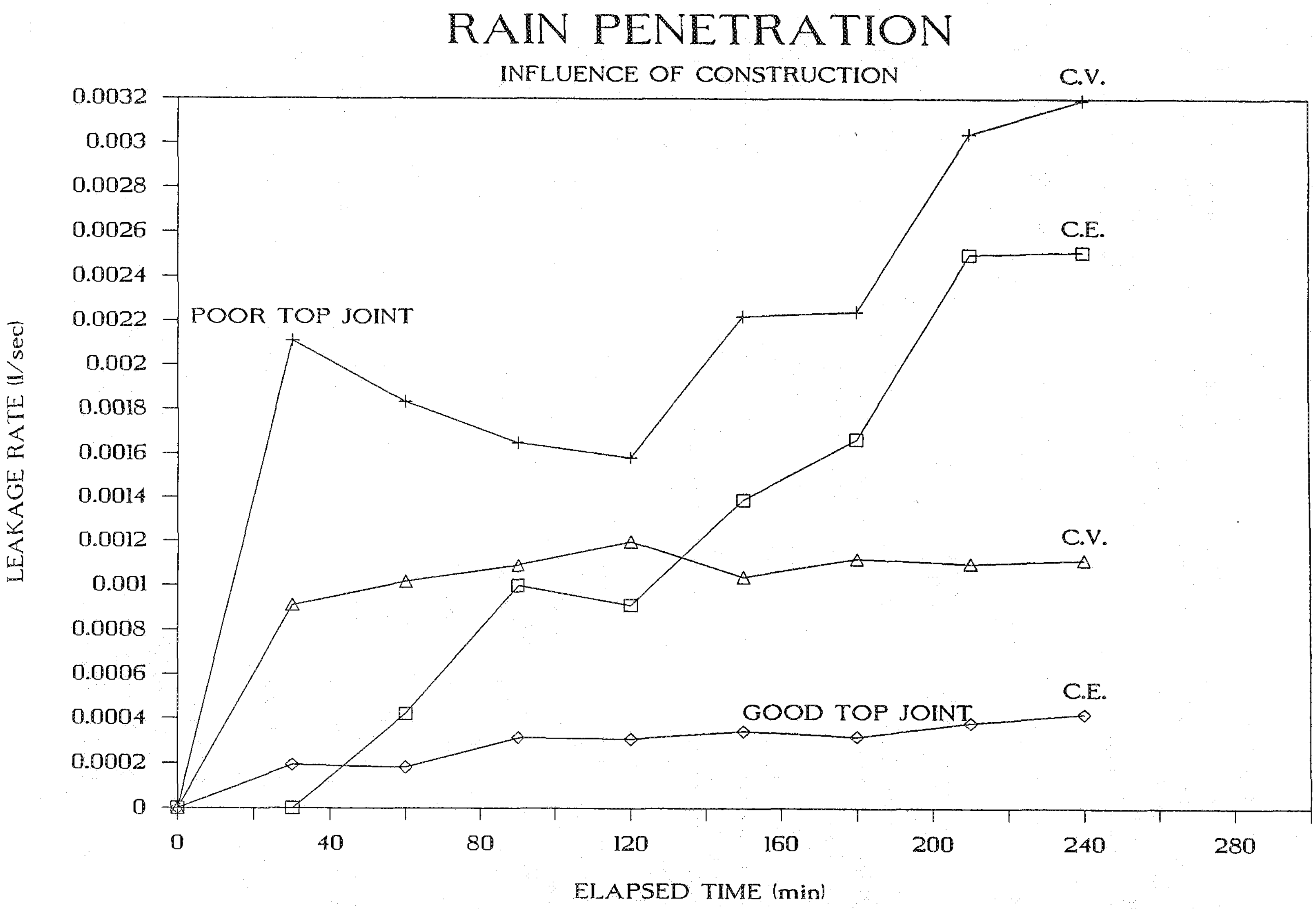



**FIGURE 22.9 INFLUENCE OF CAVITY PRESSURE EQUALIZATION AND QUALITY OF TOP JOINT ON RAIN PENETRATION OF TEST WALL 2**

## 22.4 OVERALL SUMMARY AND CONCLUSION

### 22.4.1 Test Apparatus

A unique test facility designed for structural and rain penetration research has been constructed and has been shown to be suited to the current research program. Materials, equipment and man hours which went into constructing this test apparatus gives a replacement value of approximately $50,000 exclusive of design or supervisory time.

### 22.4.2 Structural Performance of BV/SS Wall Systems

The test apparatus has proven ability to test walls under closely controlled conditions using electronically monitored data systems. The latter allow readings to be taken at high rates so that transient influences can be recorded.

The structural tests provided data which agrees quite closely with computer analysis of the same wall systems. The results confirm that cracking of brick veneer can be expected to occur for high wind loads. Also the unequal loading of ties was demonstrated and experimentally determined values are available.

### 22.4.3 Rain Penetration

The influence of air tightness on the backup wall, development of a pressurized cavity, and the effect of cracks in the brick veneer have been investigated. The results confirm the need to ensure pressure equalization of cavities. It was also shown that cracked brick veneer is much more vulnerable to excessive rain penetration where cavity pressure equalization is not achieved.

PART 4

# DEFINING BETTER CLADDING SYSTEMS - THEORETICAL WORK

Summary of a Report

Submitted to the

RESEARCH DIVISION

CANADA MORTGAGE AND HOUSING CORPORATION
OTTAWA, ONTARIO

By

R. G. Drysdale, Ph.D., P.Eng. & S. E. Chidiac, Ph.D.

## CHAPTER 23
## DEFINING BETTER CLADDING SYSTEMS - THEORETICAL WORK

### 23.1 INTRODUCTION

The primary objective of this project is to develop new load and load distribution recommendations to be used by designers in the design of MV/SS walls in high rise buildings. The major effort to date has been on the development of the analytical models. While only this topic is dealt with in this summary report, some other activities are included in the full CMHC report.

### 23.2 ANALYSIS OF DESIGN ALTERNATIVES

Attempts have been made to critically evaluate the use of masonry veneer and steel stud backup wall construction with the view to developing new concepts for design with these structural components. As a result, using modelling of two-dimensional behaviour, two different concepts have been analyzed.

i) Use of the Brick Veneer as the Structural Element Resisting Wind Pressure:

The use of post-tensioned brick veneer as the wind resisting element has been found to be a structurally feasible concept which within itself has several options. Using tie systems between the veneer and steel stud backup wall, the steel studs transfer wind pressure from cavity pressure equalization and internal building pressures back to the veneer which reacts against the floor below and floor or roof above to transfer the forces to the main structure. In this concept the studs span between ties and can be substantially down sized. Besides transferring force back to the veneer, they act as the platform for the insulation, air, and vapour barriers.

An alternate concept which doesn't require much analysis is to design the veneer and steel stud layers to act independently. That is, each is supported top and bottom without tying together at intervals over the height. To the degree that pressure equalization of the cavity with the external air pressure occurs, the brick veneer is relieved of wind loading. Nonetheless, prestressing or just normal reinforcing is necessary to provide safety against collapse. In this situation the backup wall resists the entire wind load but is free to deflect to the acceptable limit (i.e. L/360) without causing distress in the veneer.

The use of prefabricated panels is a feasible variation for both of these concepts.

ii) Modifications to the Existing Forms of Construction.

Some of the concerns about the existing forms of MV/SS construction can be significantly reduced by changes in the design approach. An idea which is currently being tested experimentally at McMaster University is to build the brick veneer prior to placing the insulation, air and vapour barriers, and internal wall covering. Thus with only the steel studs and tracks in place the veneer can be built and tied to the studs. This allows the cavity to be inspected and cleaned so that water which penetrates to the cavity can flow out through weep holes which are guaranteed to be open. In addition, the existence and proper installation of ties can be inspected prior to filling in of the other components of the steel stud backup wall. This concept requires that both the air barrier and the vapour barrier be located on the interior face of the steel stud wall. Additional feature of this redesigned wall is the positioning of the movement joint at the bottom of the wall and an improved system of surface bridging for torsional stiffening of the steel studs.

Another concept which has been reviewed, is the idea of constructing a predetermined mid-height crack in the brick veneer. This idea was presented by Proen Consultants in a previous CMHC study. While it may be difficult to persuade designers to use this concept, caulking of the crack has the potential for reducing the expected increases in water penetration associated with use of flexible backup for masonry veneer.

Preliminary structural analyses of the above design concepts have been completed and many of the practical construction details and methods for integrating the thermal, moisture, and air barriers have been developed. These concepts will be discussed in detail with industrial representatives and reported on fully in the future.

## 23.3 PREPARATION OF THREE DIMENSIONAL MODELLING OF MV/SS SYSTEM

### 23.3.1 *General Description*

Most of the research effort has been devoted to development of a model for three dimensional behaviour of the MV/SS system. While the study of two dimensional behaviour is very useful to developing an understanding of the influence of various design parameters, the existence of wall openings and various boundary conditions along the sides of walls necessitates modelling of three dimensional behaviour. The finite element model has the following components:

1. Non-Conforming Plate Bending Elements:
   These elements are intended mainly to model the masonry veneer but may also be employed to model drywall or other sheathing attached directly to the steel studs. The elements have 4 nodes with 5 degrees of freedom per node. This provides a thin plate type of model which excludes shear distortion. Initially the model has been developed for isotropic homogenious elastic properties but will be modified in the future to include orthotropic stiffness properties. As will be discussed below, orthotropic strength characteristics have been included in the formulations of the failure criteria.

2. Grid Elements:
   The steel studs and cross members of the steel stud backup wall are modelled using 2 node grid elements with 3 degrees of freedom at each node including twisting (torque). These one dimensional elements can be placed at any orientation. The full range of end connections and interconnections of the grid elements (steel stud components) can be accommodated in each degree of freedom.

   The modelling will force the two common nodes at the ends of grid elements to behave identically in all or part of the degrees of freedom.

3. Spring Elements:
   The ties can be modelled using spring elements with five degrees of freedom where all or any one of these may have specified stiffnesses.

### 23.3.2 Mathematical Modelling Details

**Discretized Equations of Equilibrium**

To establish the discretized equations of equilibrium of linear elastic systems, the potential energy theorem can be used and is defined by

$$\pi = U - W \qquad (1)$$

Where $\pi$ is the potential energy, U is the strain energy and W is the work done by the external loads. Equation 1 can be rewritten as

$$\pi = 1/2 \int_v \sigma_{ij} \varepsilon_{ij}\, dv - (\int_v F_i u_i\, dv + \int_{s_t} T_i U_i\, ds) \qquad (2)$$

where

$u_i$ = displacement

$\varepsilon_{ij}$ = strain

$\sigma_{ij}$ = stress

$F_i$ = body force

$T_i$ = surfacetraction

However, for the system to be in equilibrium, the potential energy must be stationary due to variations of all possible displacements. This implies that

$$\frac{\partial \pi}{\partial u_i} = 0 \qquad (3)$$

The applications of Equation 3 to Equation 2 leads to the equilibrium equations that correspond to all possible displacements in a finite element analysis. After integrating, the equations are given by

$$\int_v \varepsilon_{ij} \sigma_{ij}\ dv = \int_v u_i f_i\, dV + \int_s u_i T_i\ ds \qquad (2)$$

The continuum is divided into a finite number of subdomains called finite elements. These finite elementsare used to approximate the response of the continuum and are interconnected at the nodal points on the element boundaries. The displacements at any point inside an element are related to the nodal displacements through the displacement field assumed over an element, and is given by

$$\{u_i\} = [N]\{\delta\} \qquad (2)$$

where

[N] = matrix of shape functions

$\{\delta\}$ = nodal displacement degree of freedom

Thus the stress and strain fields within an element can be obtained in the following manner;

$$\{\varepsilon\} = [B]\{\delta\} \quad (6)$$
$$\{\sigma\} = [D]\{\varepsilon\} \quad (7)$$

where

$[B]$ = strain matrix relating strains to nodal degrees of freedom

$[D]$ = stress strain constitutive matrix.

Substitutions of Equations 5, 6 and 7 into Equation 4 and applying Equation 3 yields

$$[K]\{\delta\} = \{F\} \quad (8)$$

where
$$[K] = \int_v [B]^T [D][B]\, dv \quad (9)$$
$$\{F\} = \Sigma \int_v \{f\}\, dv + \Sigma \int_v \{T\}\, ds \quad (10)$$

**Finite Element**

**1. Rectangular Plate Element**

The rectangular element used in this study combines the plane stress element for in plane actions with the non-conforming plate bending element for out-of-plane action. The rectangular element, therefore, has five degree of freedom per node namely $u_i$, $v_i$, $w_i$, $w_{xi}$ and $w_{yi}$ and twenty degrees of freedom per element. It should be noted that $w_{xi}$ is the derivative of w with respect to x at node i and $w_{yi}$ is the derivative of w with respect to y at node i.

The element displacement field can be written in terms of the nodal degrees of freedom through use of shape functions, i.e.

$$\{U\} = [N]\{\delta\} \quad (11)$$

where

$$\{U\} = \begin{Bmatrix} u \\ v \\ w \end{Bmatrix} \quad \text{and} \quad \begin{aligned} u &= \sum_{i=1}^{4} N_i u_i \\ v &= \sum_{i=1}^{4} N_i v_i \\ w &= \sum_{i=1}^{12} \overline{N}_i w_i \end{aligned} \tag{12}$$

and

$$\{\delta\}^T = \left\langle u_1, v_1, w_1, \frac{\partial w_1}{\partial x}, \frac{\partial w_1}{\partial y}, u_2, v_2, w_2, \frac{\partial w_2}{\partial x}, \frac{\partial w_2}{\partial y}, \ldots \right\rangle \tag{13}$$

**Ni are the in-plane action shape functions while $\overline{N}_i$ are those for bending.**

From geometry, the displacement field is given by:

$$\begin{aligned} u^T &= u(x,y) - z\frac{\partial w(x,y)}{\partial x} \\ v^T &= v(x,y) - z\frac{\partial w(x,y)}{dy} \\ w^T &= w(x,y) \end{aligned} \tag{14}$$

Now assuming small displacements, the strain components are given by:

$$\begin{aligned} \varepsilon_x &= \frac{\partial u^T}{\partial x} = \frac{\partial u}{\partial x} - z\frac{\partial^2 w}{\partial x^2} \\ \varepsilon_y &= \frac{\partial v^T}{\partial y} = \frac{\partial v}{\partial y} - z\frac{\partial^2 w}{\partial y^2} \end{aligned} \tag{15}$$

$$\gamma_{xy} = \frac{\partial u^T}{\partial y} + \frac{\partial y^T}{\partial x} = \frac{\partial u}{\partial y} + \frac{\partial v}{\partial x} - 2z\frac{\partial^2 w}{\partial x \partial y}$$

in matrix form

$$\{\varepsilon\} = [L]\{U\}$$

$$= \begin{bmatrix} \frac{\partial}{\partial x} & 0 & -z\frac{\partial^2}{\partial x^2} \\ \frac{\partial}{\partial y} & 0 & -z\frac{\partial^2}{\partial y^2} \\ \frac{\partial}{\partial y} & \frac{\partial}{dx} & -2z\frac{\partial^2}{\partial x \partial y} \end{bmatrix} \begin{bmatrix} u \\ V \\ W \end{bmatrix} \qquad (16)$$

Substituting Equation 12 into equation 16 yields

$$\begin{aligned} \{\varepsilon\} &= [L]\ \{U\} \\ &= [L]\ [N]\ \{\delta\} \\ &= [B]\ \{\delta\} \end{aligned}$$

$$\begin{bmatrix} \mathcal{E}_1 \\ \mathcal{E}_2 \\ \mathcal{E}_3 \end{bmatrix} = \begin{bmatrix} \frac{\partial N_1}{\partial x} & 0 & -z\frac{\partial^2 \bar{N}_1}{\partial x^2} & -z\frac{\partial^2 \bar{N}_2}{\partial x^2} & \cdots \\ 0 & \frac{\partial N_1}{\partial y} & -z\frac{\partial^2 \bar{N}_1}{\partial y^2} & -z\frac{\partial^2 \bar{N}_2}{\partial y^2} & \cdots \\ \frac{\partial N_1}{\partial y} & \frac{\partial N_1}{\partial x} & -2z\frac{\partial^2 \bar{N}_1}{\partial x \partial y} & -2z\frac{\partial^2 \bar{N}_2}{\partial x \partial y} & \cdots \end{bmatrix} \begin{bmatrix} u_1 \\ v_1 \\ \frac{\partial w_1}{\partial x} \\ \frac{\partial w_1}{\partial y} \\ \vdots \end{bmatrix} \qquad (17)$$

Having obtained the strain matrix [B] relating nodal displacement to strains. The stiffness matrix can now be completed using

$$[K] = \int_V [B]^T [D] [B] \, dV \qquad (18)$$

To integrate the stiffness matrix, Gaussian quadrature is used with three integration points in each direction. One needs to transform the equation from global x, y and z coordinates to a non-dimensional one namely r, s and t using;

$$x = \frac{a}{2}(s+1)$$

$$y = \frac{b}{2}(t+1) \qquad (19)$$

$$z = \frac{t_0}{2} r$$

Substituting Equation 19 into Equation 18 yields:

$$[K] = \frac{a}{2}\frac{b}{2}\frac{t_0}{2} \int\int\int [B]^T [D] [B] \, ds \, dt \, dr \qquad (20)$$

Grid Element

**The grid element is also used in this study which only takes the torsional and bending stiffnesses of the member into consideration. The element has two nodes with three degree of freedom per node, namely w, $w_x$ and $w_y$. The grid element is a much simpler element to compute and its stiffness matrix is already available in closed form and is given below:**

$$\left[\overline{K}_{Grid}\right] = \begin{bmatrix} GJ/L & & & & & \\ 0 & 4EI_y/L & & \text{Symmetric} & & \\ 0 & -6EI_y/L^2 & 12\,EI_y/L^3 & & & \\ -GJ/L & 0 & 0 & GJ/L & & \\ 0 & 2EI_y/L & -6EI_y/L^2 & 0 & 4EI_y/L & \\ 0 & 6EI_y/L^2 & -12EI_y/L^3 & 0 & 6EI_y/L^2 & 12EI_y/L^3 \end{bmatrix}$$

In order to compute the stiffness matrix for an arbitrary orientation, a transformation from local to global coordinate system is required. The transformed matrix [K] in the global coordinate system is computed using

$$[K] = [T]^T [\bar{K}] [T] \tag{22}$$

where

$$[T] = \begin{matrix} [R] & [\phi] \\ [\phi] & [R] \end{matrix}$$

and

$$[R] = \begin{bmatrix} \cos \phi & \sin \phi & 0 \\ -\sin \phi & \cos \phi & 0 \\ 0 & 0 & 1 \end{bmatrix} \tag{23}$$

Note that the displacement vector is given by:

$$\{\delta\} = \begin{bmatrix} Wx_1 \\ Wy_1 \\ W_1 \\ Wx_2 \\ Wy_2 \\ W_2 \end{bmatrix} \tag{24}$$

3. Spring Element

**The spring element is used in this study to simulate connections and it has five degrees of freedom and its stiffness matrix is given by:**

$$K_s = \begin{bmatrix} k_x & & & & & & & & & \\ 0 & k_y & & & & & \text{Symmetric} & & & \\ 0 & 0 & k_z & & & & & & & \\ 0 & 0 & 0 & k_{\theta x} & & & & & & \\ 0 & 0 & 0 & 0 & k_{\theta y} & & & & & \\ -k_x & 0 & 0 & 0 & 0 & k_x & & & & \\ 0 & -k_y & 0 & 0 & 0 & 0 & k_y & & & \\ 0 & 0 & -k_z & 0 & 0 & 0 & 0 & k_z & & \\ 0 & 0 & 0 & -k_{\theta x} & 0 & 0 & 0 & 0 & k_{\theta x} & \\ 0 & 0 & 0 & 0 & -k_{\theta y} & 0 & 0 & 0 & 0 & k_{\theta y} \end{bmatrix}$$

Failure Criterion

Failure Criterion adopted for the MVSS wall analysis.

i) Strength for Debonding Along Bed Joints .

$$\sigma_n = \sigma_{tbmo} \left( 1 - \frac{\tau_{np}}{\sigma_{sbmo}} \right) \tag{25}$$

where

$\sigma_n$ = Tensile stress at the extreme fibers (normal to the bed joint)

$\tau_{np}$ = shear stress at the extreme fibers

$\sigma_{sbmo}$ = shear bond strength

$\sigma_{tbmo}$ = Tensile bond strength

ii) Strength for Debonding Along Head Joints and Splitting through units in Alternate Courses

$$\sigma_p = \frac{1}{2}\left[\sigma_{tbmo}\left(1 - \frac{1 + 2E_j/G_b}{1 + \frac{E_j}{G_b} + \frac{E_j}{G_j}} \cdot \left(\tau_{np}/\sigma_{sbmo}\right)\right) + \sigma_{tbl}\right] \tag{26}$$

$\sigma_p$ = Stress parallel to the bed joint

$E_j$ = E of mortar joints

$G_j$ = G of mortar joints

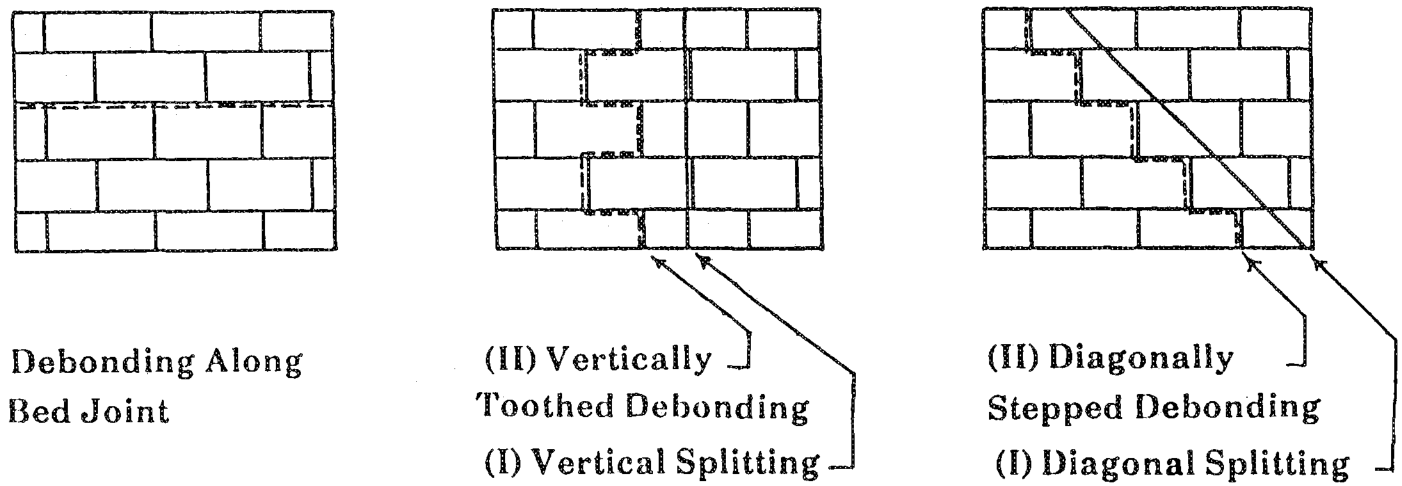
$G_b$ = G of masonry unit material

$\sigma_{tb1}$ = tensile strength of the masonry unit

iii) Strength for Debonding Along a combination of Bed and Head Joints

a) for only $\sigma_n$ in compression

$$\frac{\sigma_n}{\sigma_{sbmo}\left(\frac{b}{a} - \frac{1}{\mu}\right)} + \frac{\sigma_p}{\frac{a}{b}\sigma_{sbmo} + \sigma_{tbmo}} + \tau_{np}\left[\frac{1}{\sigma_{sbmo} + \frac{b}{a}\sigma_{tbmo}} - \frac{1}{\sigma_{sbmo}\left(1 - \frac{a}{b}\frac{1}{\mu}\right)}\right] = 1 \tag{27}$$




Possible Transverse Failure Modes

b) $\sigma_p$ in compression $\dfrac{\sigma_n}{\frac{b}{a}\sigma_{sbmo} + \sigma_{tbmo}} + \dfrac{\sigma_p}{\sigma_{sbmo}\left(\frac{a}{b} - \frac{1}{\mu}\right)}$

$$+ \tau_{np}\left[\frac{-1}{\sigma_{sbmo}\left(1 - b/a\mu\right)} + \frac{1}{\left(\sigma_{sbmo} + \frac{a}{b}\sigma_{tbmo}\right)}\right] = 1 \qquad (28)$$

2a = nominal unit length

b = nominal unit height

$\mu$ = coefficient of friction between mortar and masonry units

iv) Strength for Splitting through Masonry Unit

$$\left(\tau_{np}/\sigma_{tbl}\right)^2 = \left(1 - \sigma_n/\sigma_{tbl}\right)\left(1 - \sigma_p/\sigma_{tbl}\right) \qquad (29)$$

Verification

The parts of the model have been checked individually and together against closed form solutions. Analyses of clamped and simply supported plates under uniformly distributed load and concentrated loads have been compared to textbook solutions. Also simply supported plates attached to elastic beams of different stiffness ratios have been analysed with rigid connections and spring connections. In all cases the finite element solutions have been confirmed by these comparisons with known behaviour. In addition, plate elements supported at corners, plate elements supported by spring elements, grid elements connected by spring elements, grid elements as cantilevers with torque at the end, and alternately with transverse load at the end, grid elements simply supported with load at the center, grid elements with end moments, and plates with square openings have all been analysed and confirm the numerical stability and fundamentally accurate modelling achieved with the program.

A major part of the programming and one which requires thorough verification is the implementation of the geometric failure criteria for masonry. Various geometric configurations of veneer walls (aspect ratio and support conditions) and combinations of material properties (orthogonal strength ratios and relationships between shear and tensile strength) have been run to confirm that the various failure modes shown in the attached figures can be achieved.

Description of the Computer Program

The program is able to run efficiently on an IBM-AT type of micro computer having 640K of RAM and a 10 Meg Hard Disk Drive. However, while this thought to be the minimum level of computer required, the program is designed to also run on any mini or mainframe computer provided with a standard fortran compiler (Fortran 77).

The program compares the stresses at all integration points and using the failure criteria identifies the location and type of first cracking. The option exists to allow the wall to crack by integration point or alternatively, after the crack has started, a complete crack can be quickly generated along a predetermined path. Provision also exists for determining successive crack patterns which tend to form at specific load levels. Where pre-knowledge allows, the propagation of the crack can be accommodated by changing the boundary conditions along the crack line. Otherwise the crack must be traced by successively allowed cracks to form at integration points. The next phase of the development of the program will be to provide a less time consuming technique for tracing propagation of cracks.

Testing has been done to measure the effect of mesh size for the finite element simulation. However, since the maximum spacing of nodes must correspond to the tie spacing, there is not much opportunity to reduce the computational effort by reducing the number of elements.

At the present time the program can likely only be run by a knowledgeable user. However, if the program proves to be as useful as is expected, some thought should be given to developing a more user friendly format with Pre-Processor, Post-Processor and Data Files. This will be addressed in detail when development of the program in its current form has been completed and its potential fully demonstrated.

## 23.4 EXAMPLE ANALYSES

As an example a 5.2 m long MV/SS wall constructed with 2.6 m high studs and 2.8 m high veneer was used to perform a parametric study. The 14 cases shown in Table III showed how the behaviour of the wall differed from the standard or reference wall as geometric or physical properties were changed. Figures showing deflections of the veneer and the stud at the centre of the wall both before and after cracking show the influence of these parameters. All data in the figures is for a unit load of 1 $kN/m^2$. In addition to the deflection plots, tie loads are also shown.

The principal findings are:

Cases A and B: The behaviour of the wall is not very sensitive to orthotropy of stiffness or flexural strength.

Cases C, D and E: Doubling the stiffness of the steel studs has only a very modest influence on the behaviour of the veneer until after the first crack forms. Increasing both the stiffness and strength of the veneer increases its cracking load. Doubling the stiffness of the veneer causes it to attract a greater share of the load but doesn't change the cracking load much.

Case F: Doubling tie stiffness causes cracking at slightly lower loads.

Case G: Preventing lateral translation at the top track is very beneficial.

Case H: A wall with the veneer supported at the ends by the structural frame has an increased cracking capacity.

Case I: Increasing the length of the wall had an insignificant beneficial effect.

Case J: Higher walls crack at lower load levels.

Case K: When the veneer is supported by a tie at the floor level, the extra height beyond the floor does not contribute significantly to cracking.

Case L: If the movement joint at the top of the veneer is filled, thereby restricting lateral deflection of the veneer, higher bending stresses result.

Case M: Wind load on the steel stud backup wall is a less critical situation compared to pressure applied on the veneer.

Tie Loads: In general top tie loads were quite high prior to cracking of the veneer. After cracking, ties in the region of the crack were most highly loaded.

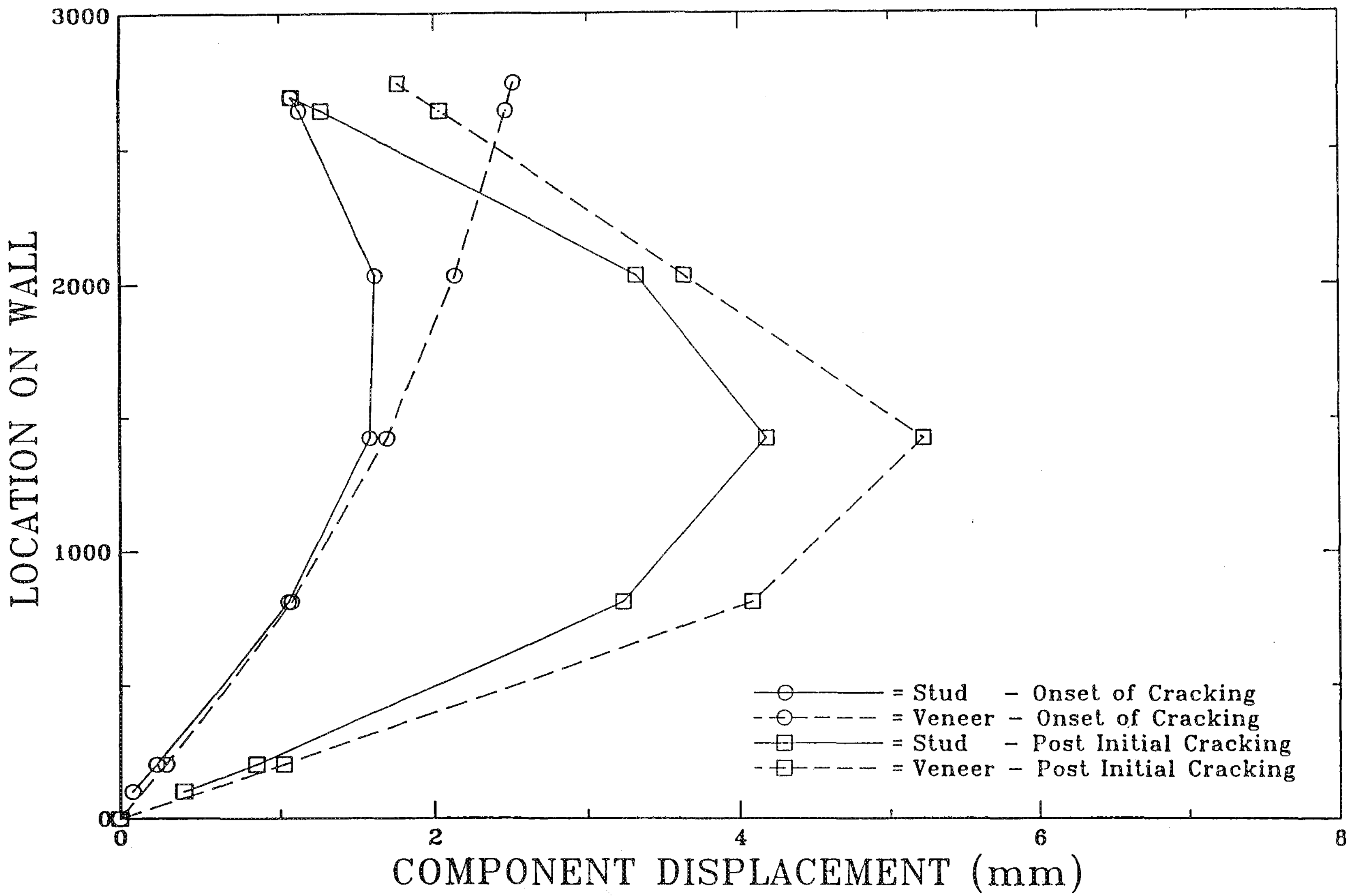
OtherAnalyses:

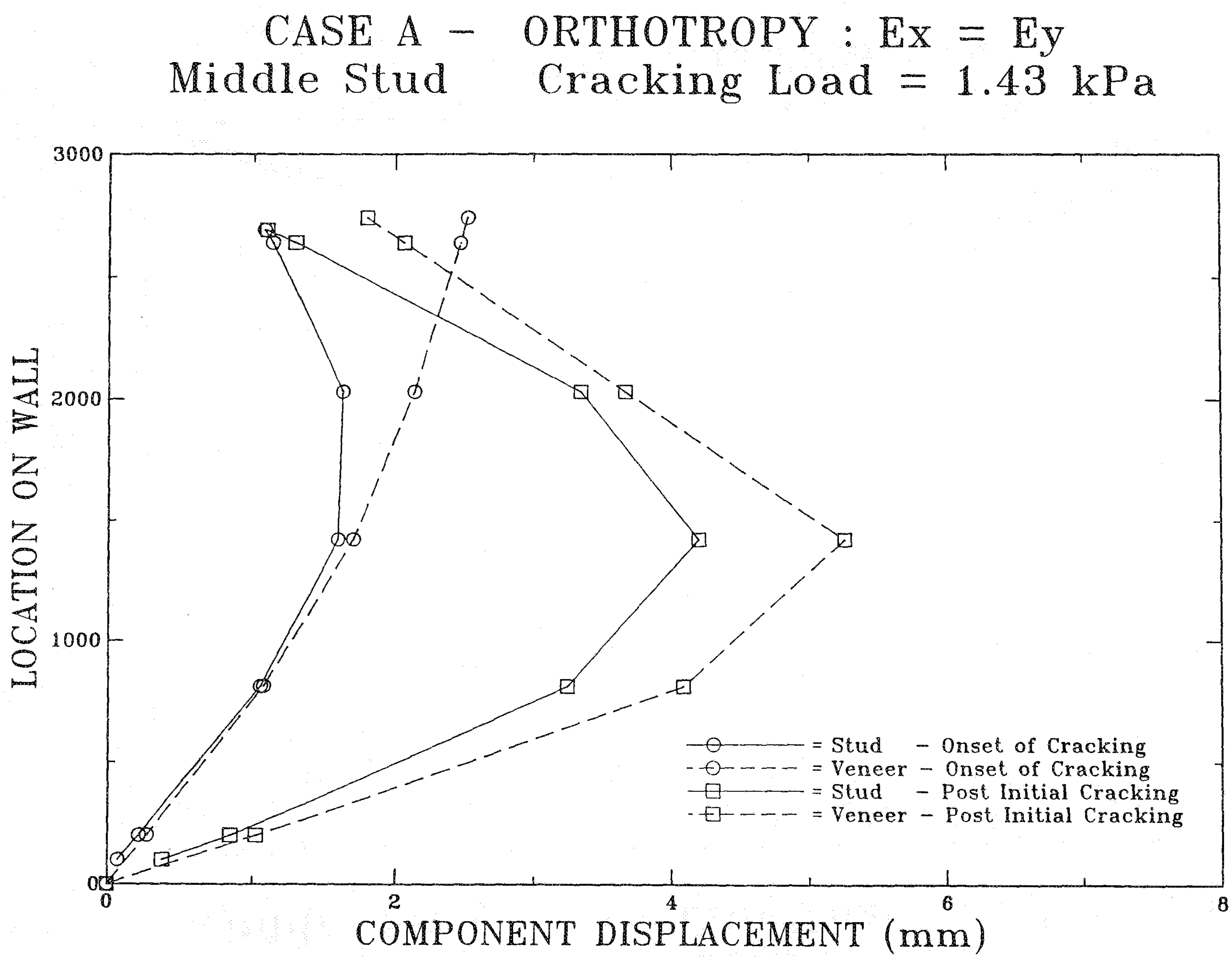
Several other analyses have been run. Of particular interest is a case where a 3.6 m x 1.8 m window was placed in the center of the wall system described above. In this case first cracking occurred at near 0.5 $kN/m^2$ regardless of changes in stiffness of the backup. Support of this type of wall at the ends increased the cracking load to 0.75 $kN/m^2$. However a second crack occurred at the same load level. The location of load transfer from the window to the supporting steel stud frame had about a 20% effect on cracking load. Since most MV/SS walls in residential type buildings have many openings, the results from such wall analyses are probably the most significant. They reveal a serious problem which is not alleviated to any great extent by doubling or tripling the studs around openings. The main benefit of increasing the stiffness of the steel stud backup is that crack width is reduced.

# TABLE III: Cracking Load For Case No. 5

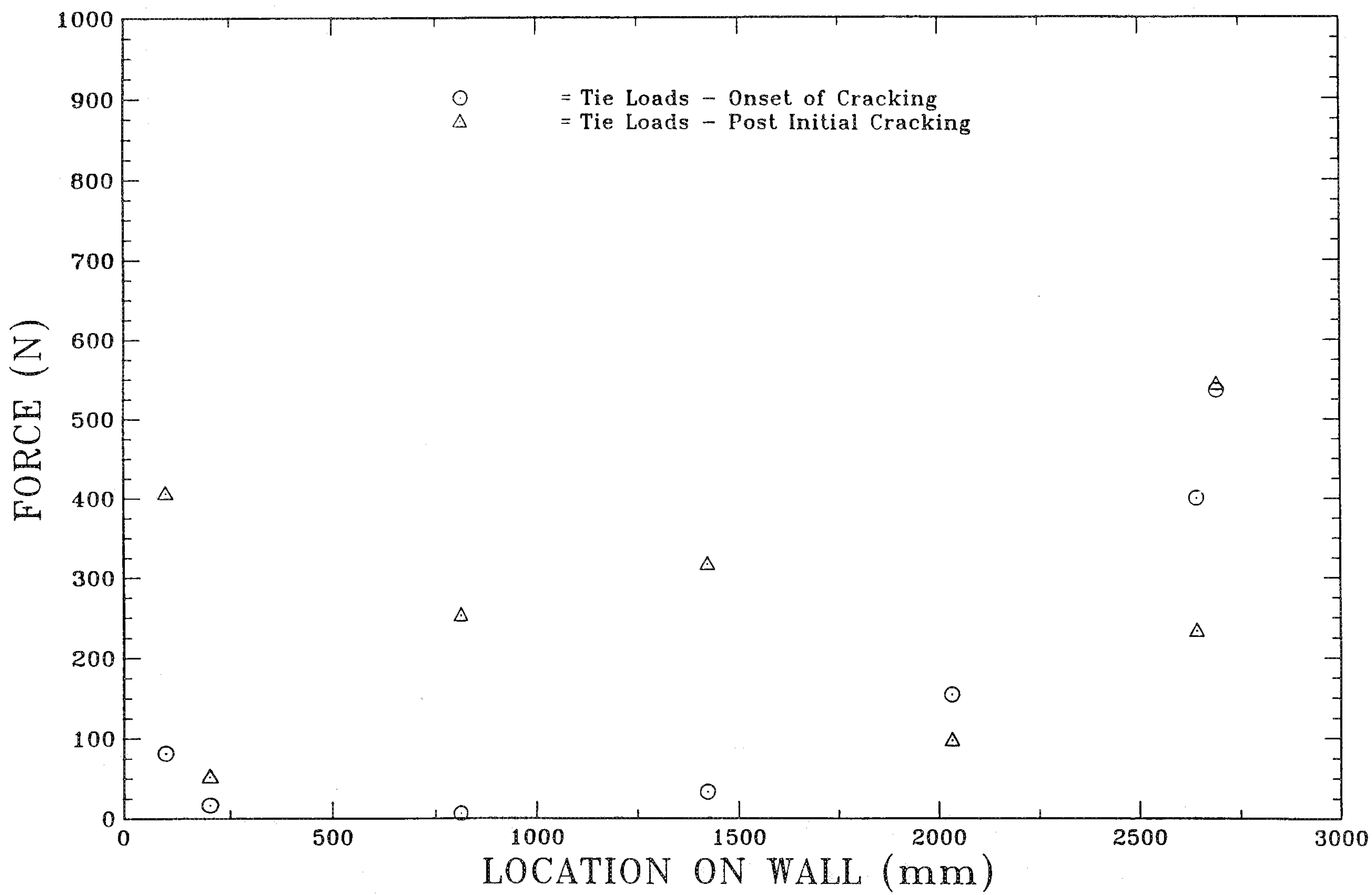
| Cases | Descriptions | First Cracking Load KN/m**2 | Second Cracking Load KN/m**2 |
|---|---|---|---|
| | Reference | 1.43 | 7.32 |
| A | Orthotropy - Ex = Ey | 1.43 | 7.58 |
| B | Orthotropy - Ftx = Fty | 1.39 | 7.28 |
| C | Double EI of S.S. | 1.69 | 11.93 |
| D | Thickness of Veneer = 140.0 mm | 3.20 | |
| E | Double E of masonry | 1.36 | 7.21 |
| F | Double the tie stiffness | 1.31 | 4.55 |
| G | Set track stiffness of infinity | 2.25 | 22.75 |
| H | Fix edge of Veneer | 1.81 | 4.02 |
| I | Geometry - aspect ratio - 2.45:1 (h constant) | 1.47 | |
| J | Geometry - height of Veneer = 3251.2 mm | 1.09 | |
| K | Geometry - relative height of Veneer & S.S. h = 3.25m, M.V. = S.S. + 18" | 1.40 | |
| L | Spring at the top of Veneer | 1.21 | 5.05 [1.1-1.2] |
| M | Load applied to the studs | 1.68 | 10.32 |

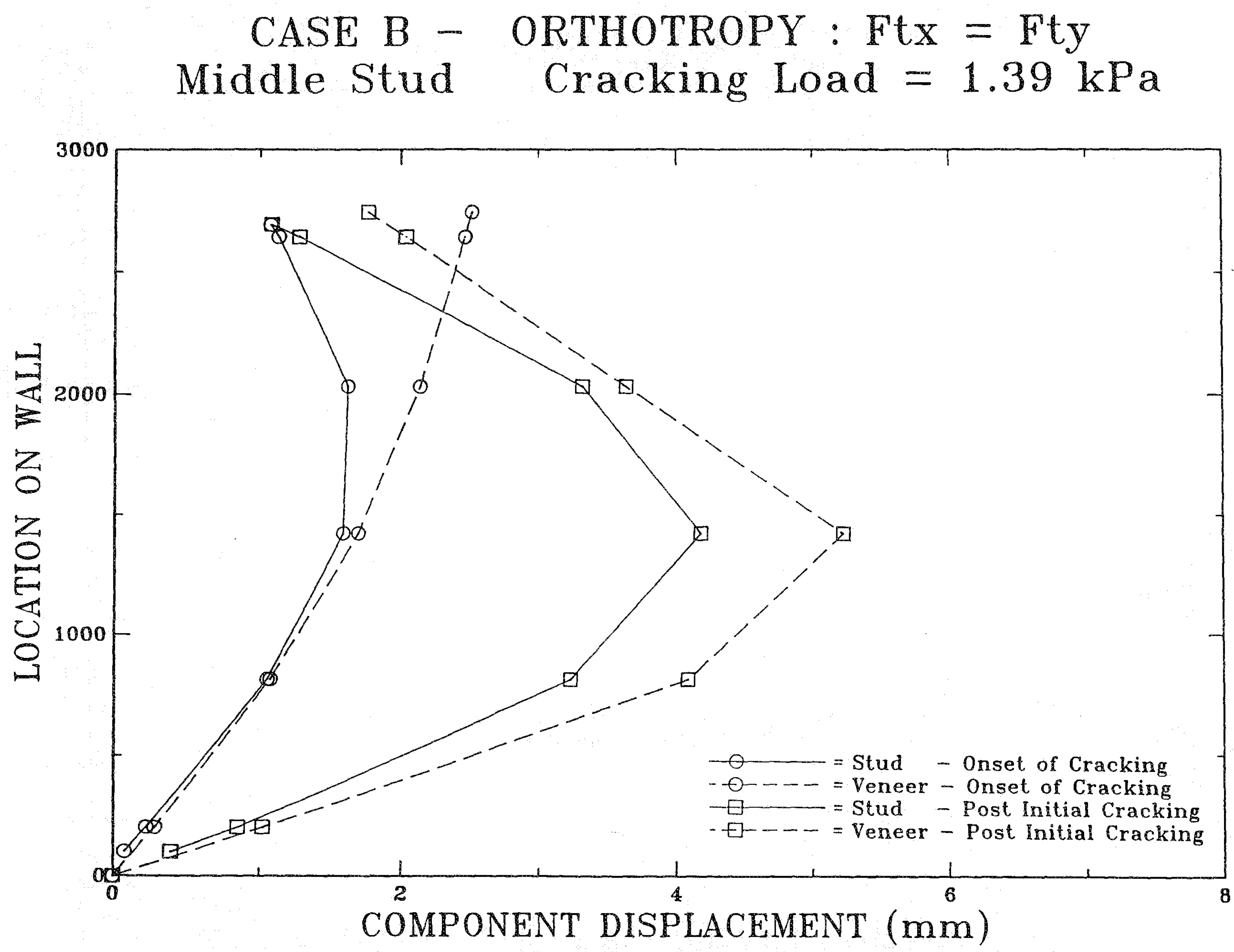
# CASE 5 REFERENCE WALL
## Middle Stud    Cracking Load = 1.426 kPa



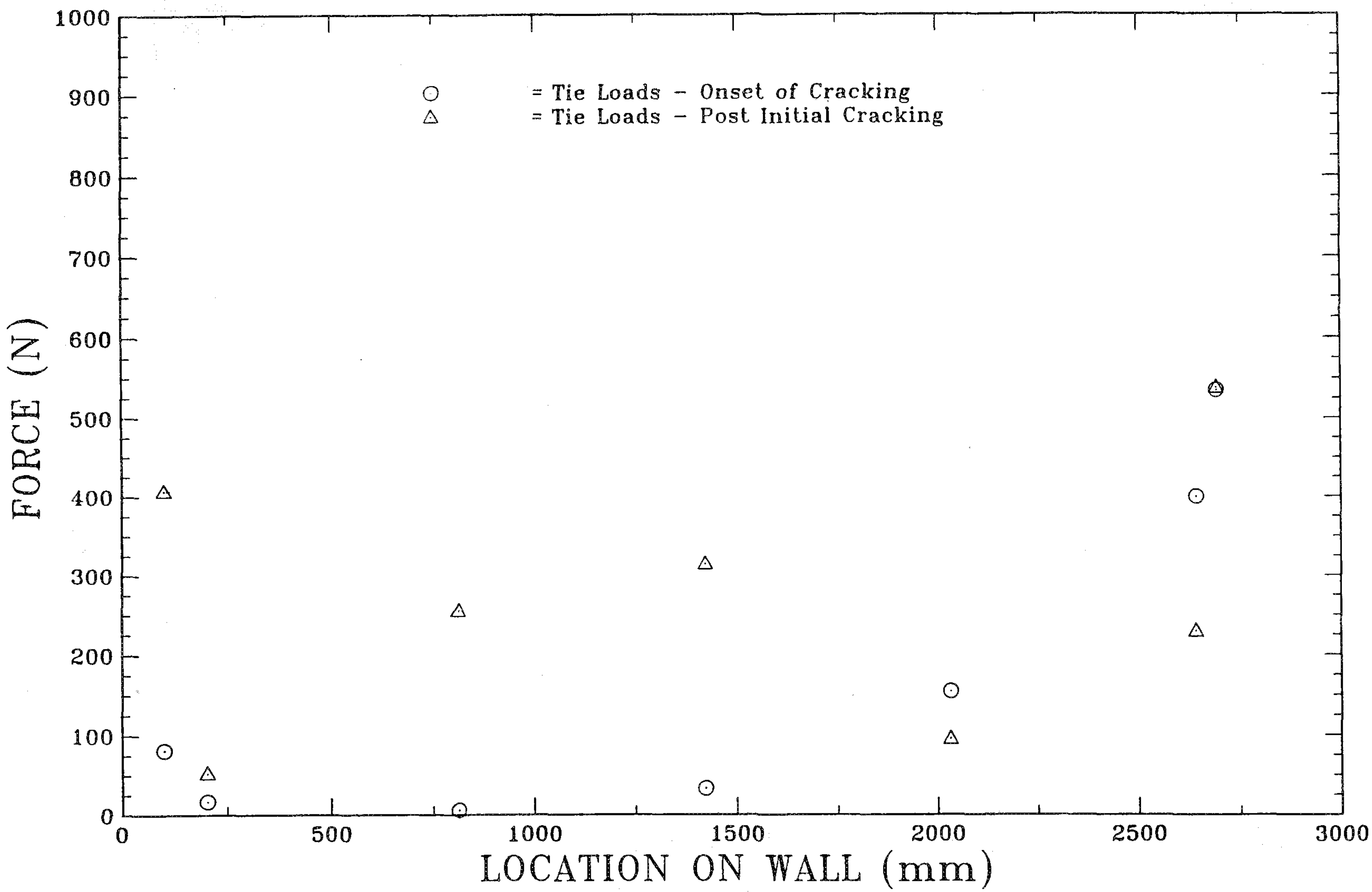

CASE A - ORTHOTROPY : Ex = Ey
Middle Stud Cracking Load = 1.43 kPa
LOCATION ON WALL
3000
2000
1000
0
0
2
4
6
8
COMPONENT DISPLACEMENT (mm)
= Stud - Onset of Cracking
= Veneer - Onset of Cracking
= Stud - Post Initial Cracking
= Veneer - Post Initial Cracking

# CASE 5A - TIE AND TRACK REACTIONS

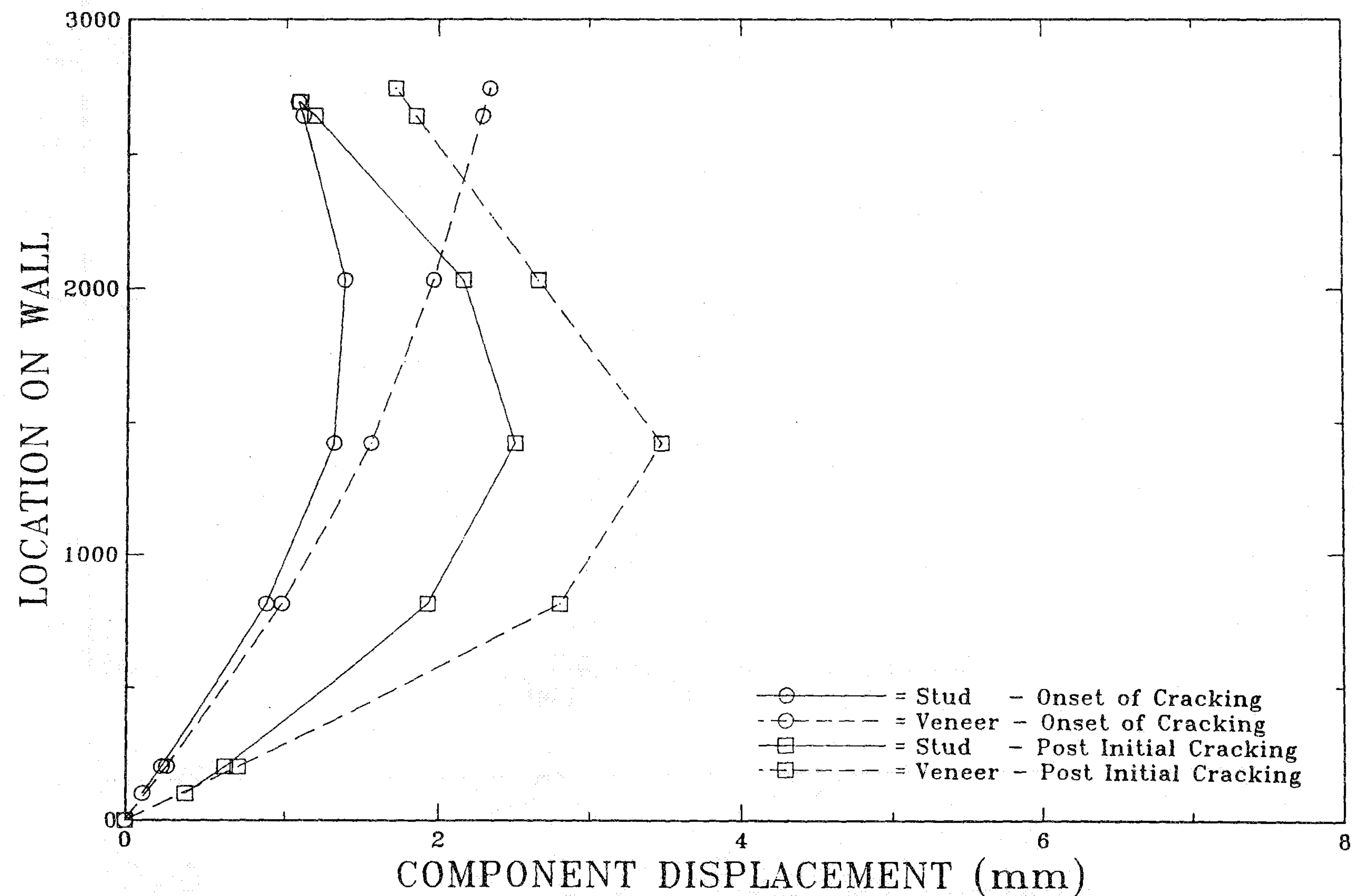


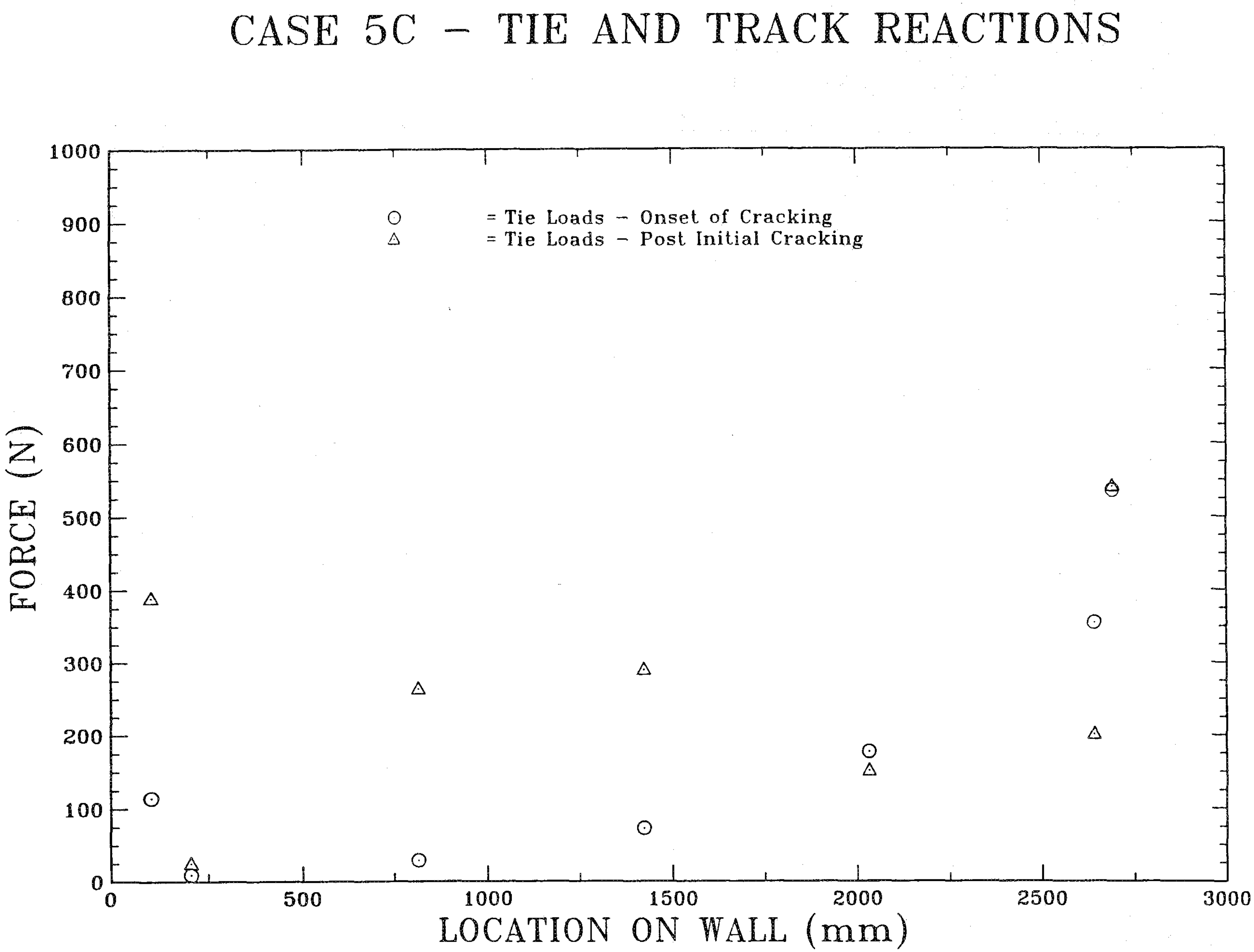

CASE B - ORTHOTROPY : Ftx = Fty
Middle Stud Cracking Load = 1.39 kPa
LOCATION ON WALL
COMPONENT DISPLACEMENT (mm)
3000
2000
1000
0
0
2
4
6
8
= Stud - Onset of Cracking
= Veneer - Onset of Cracking
= Stud - Post Initial Cracking
= Veneer - Post Initial Cracking

# CASE 5B – TIE AND TRACK REACTIONS

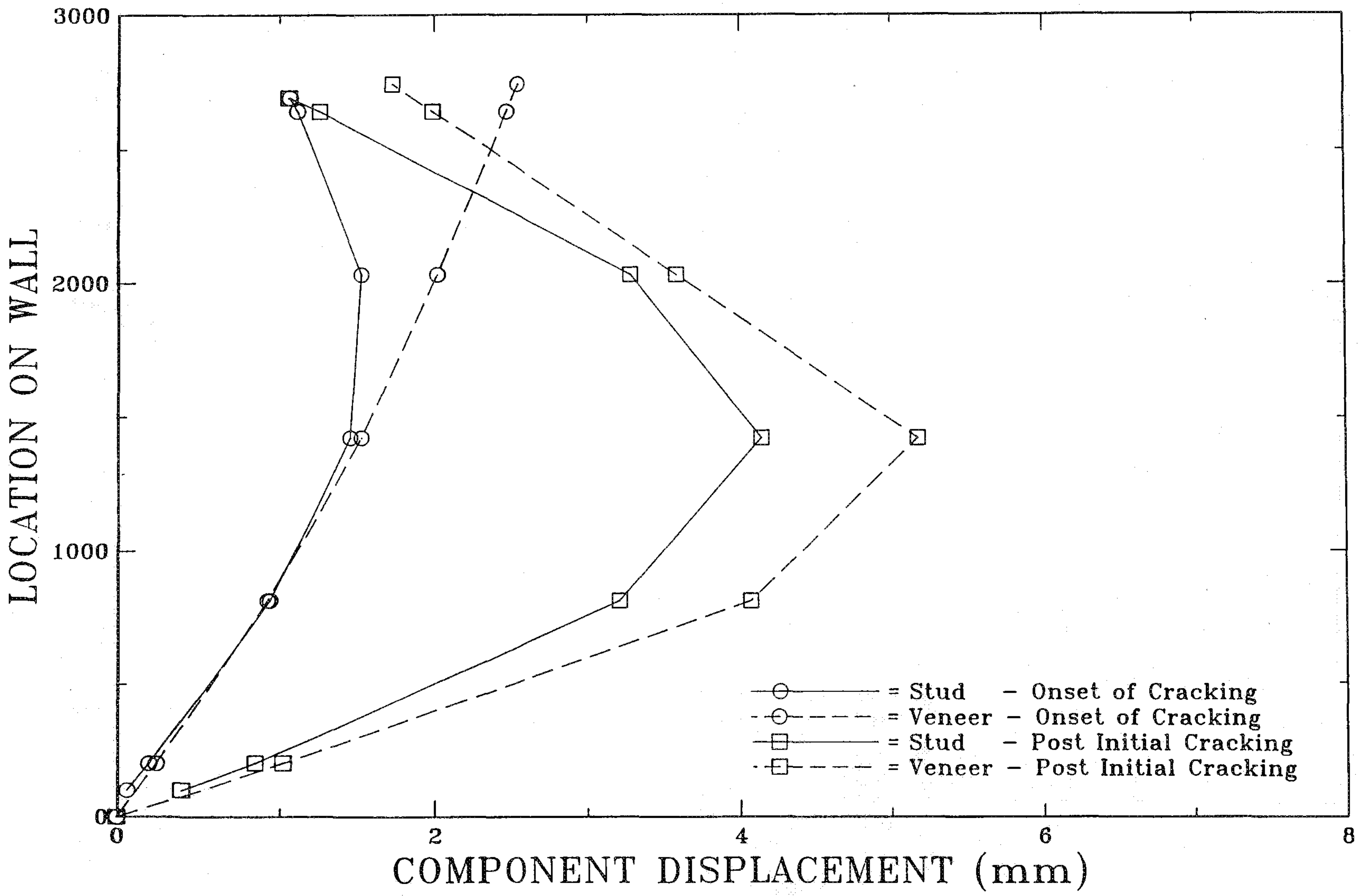


CASE C - DOUBLE EI OF STEEL STUD
Middle Stud Cracking Load = 1.69 kPa

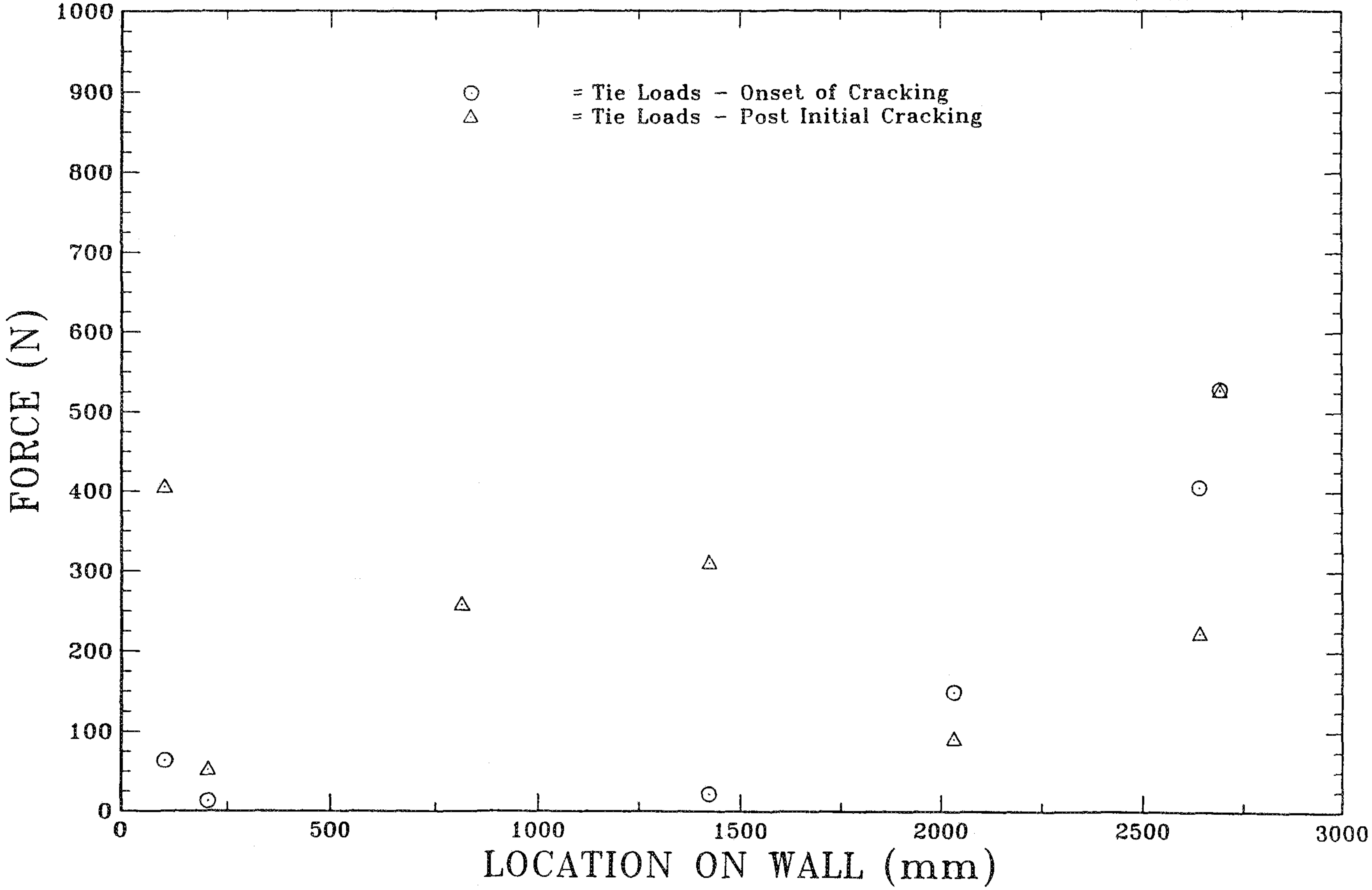



CASE 5C – TIE AND TRACK REACTIONS
= Tie Loads – Onset of Cracking
= Tie Loads – Post Initial Cracking
FORCE (N)
LOCATION ON WALL (mm)
0
100
200
300
400
500
600
700
800
900
1000
500
1000
1500
2000
2500
3000

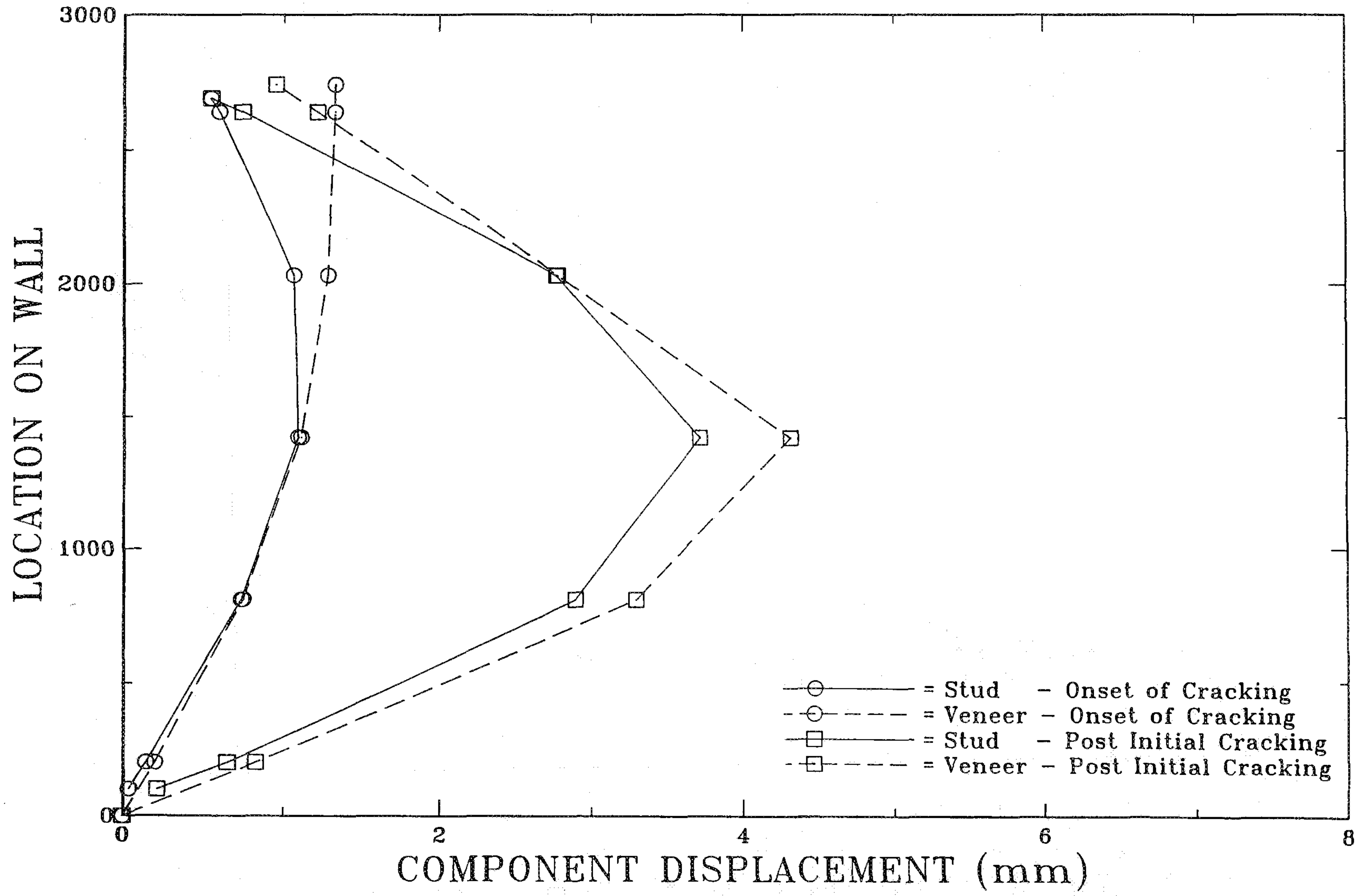
CASE E - DOUBLE EI OF MASONRY
Middle Stud Cracking Load = 1.36 kPa



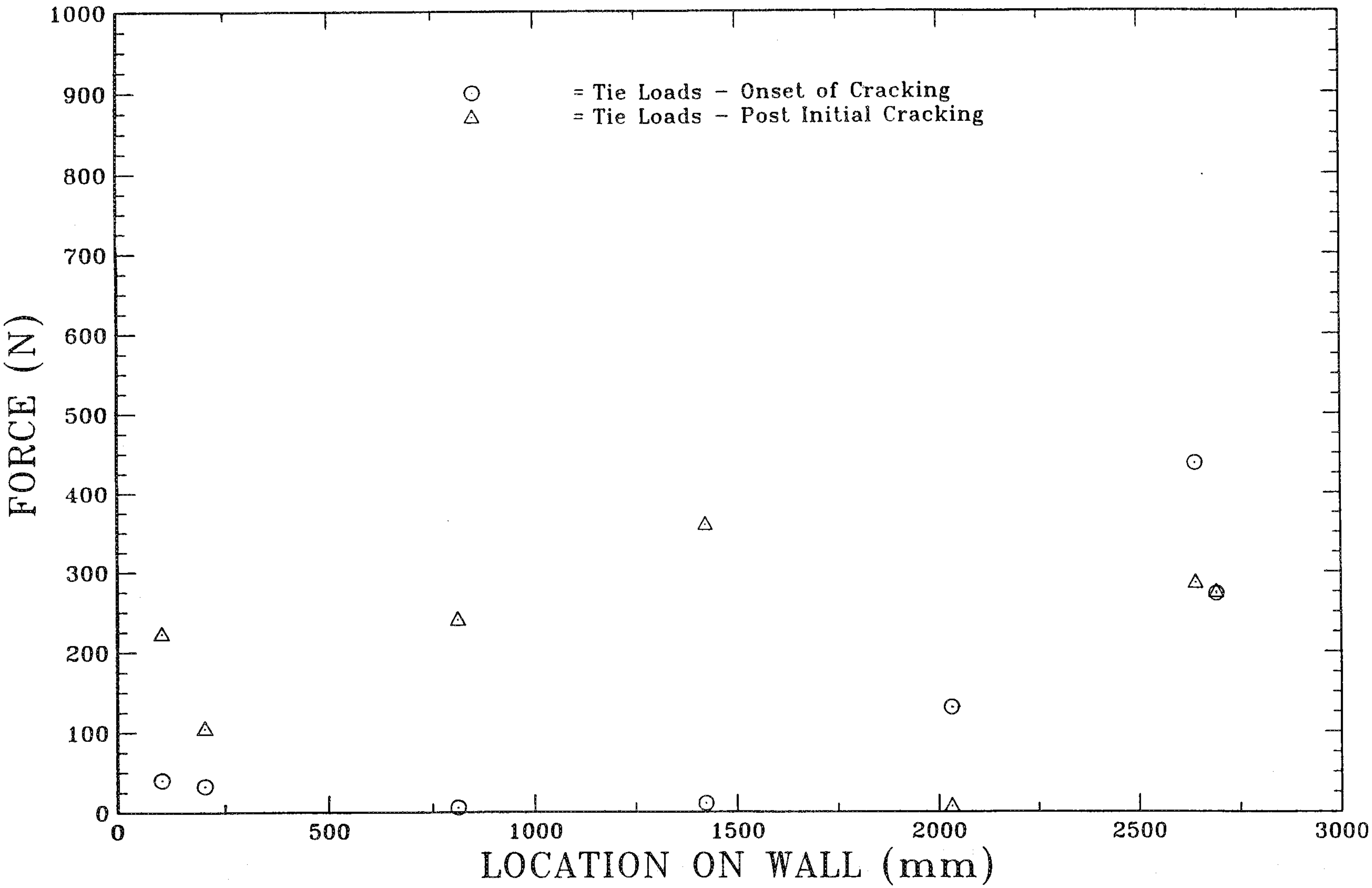
## CASE 5E – TIE AND TRACK REACTIONS

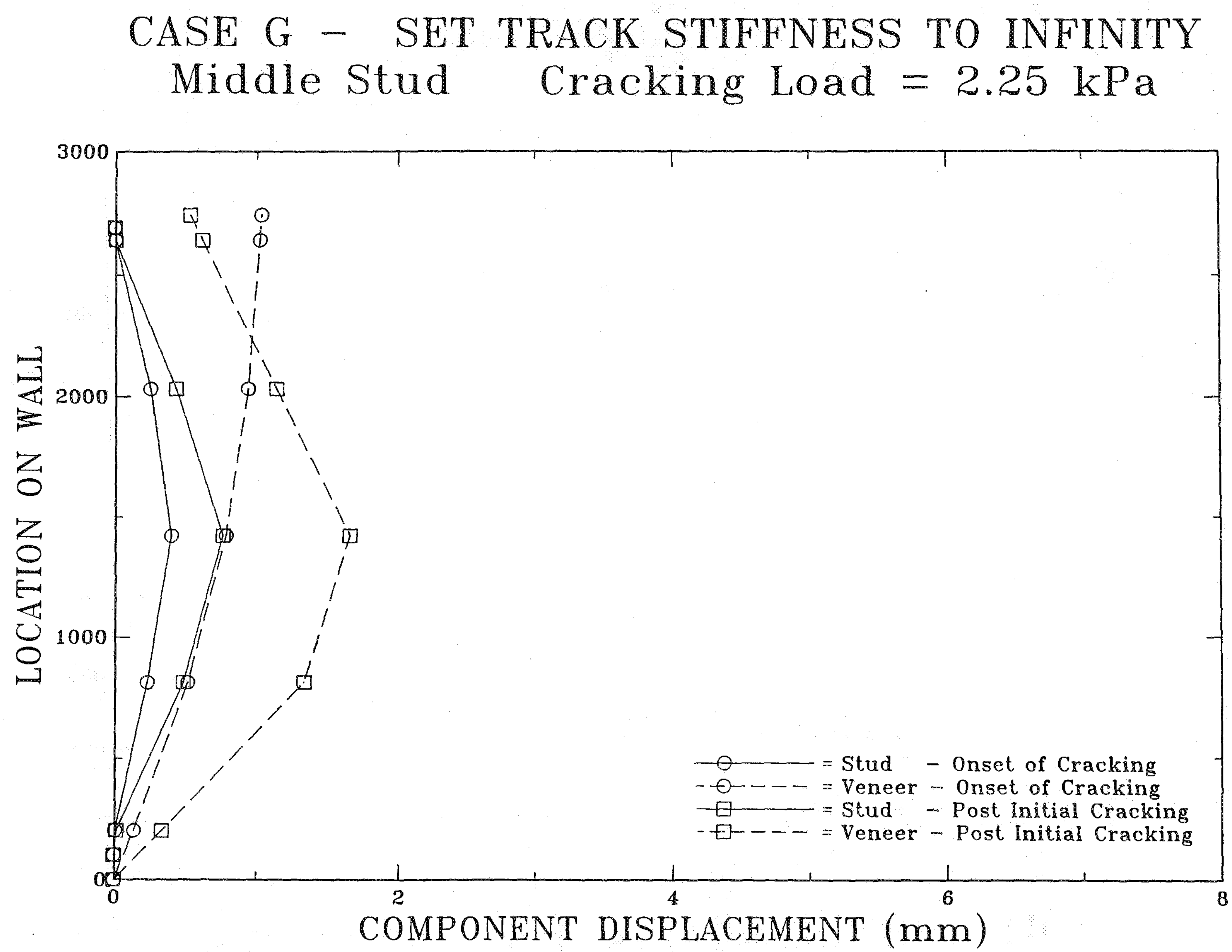


# CASE F - DOUBLE THE TIE STIFFNESS
## Middle Stud    Cracking Load = 1.31 kPa

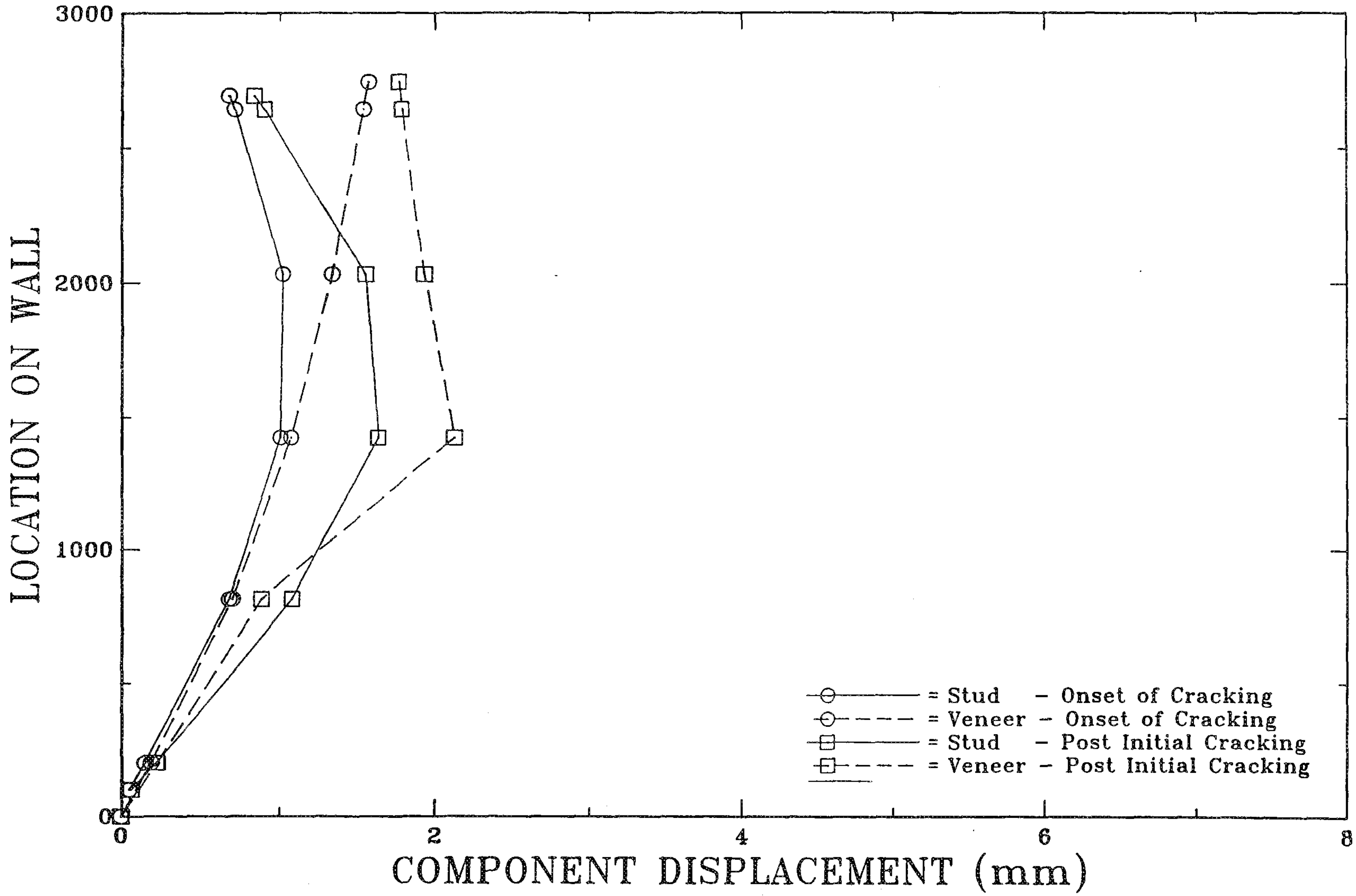


CASE 5F – TIE AND TRACK REACTIONS

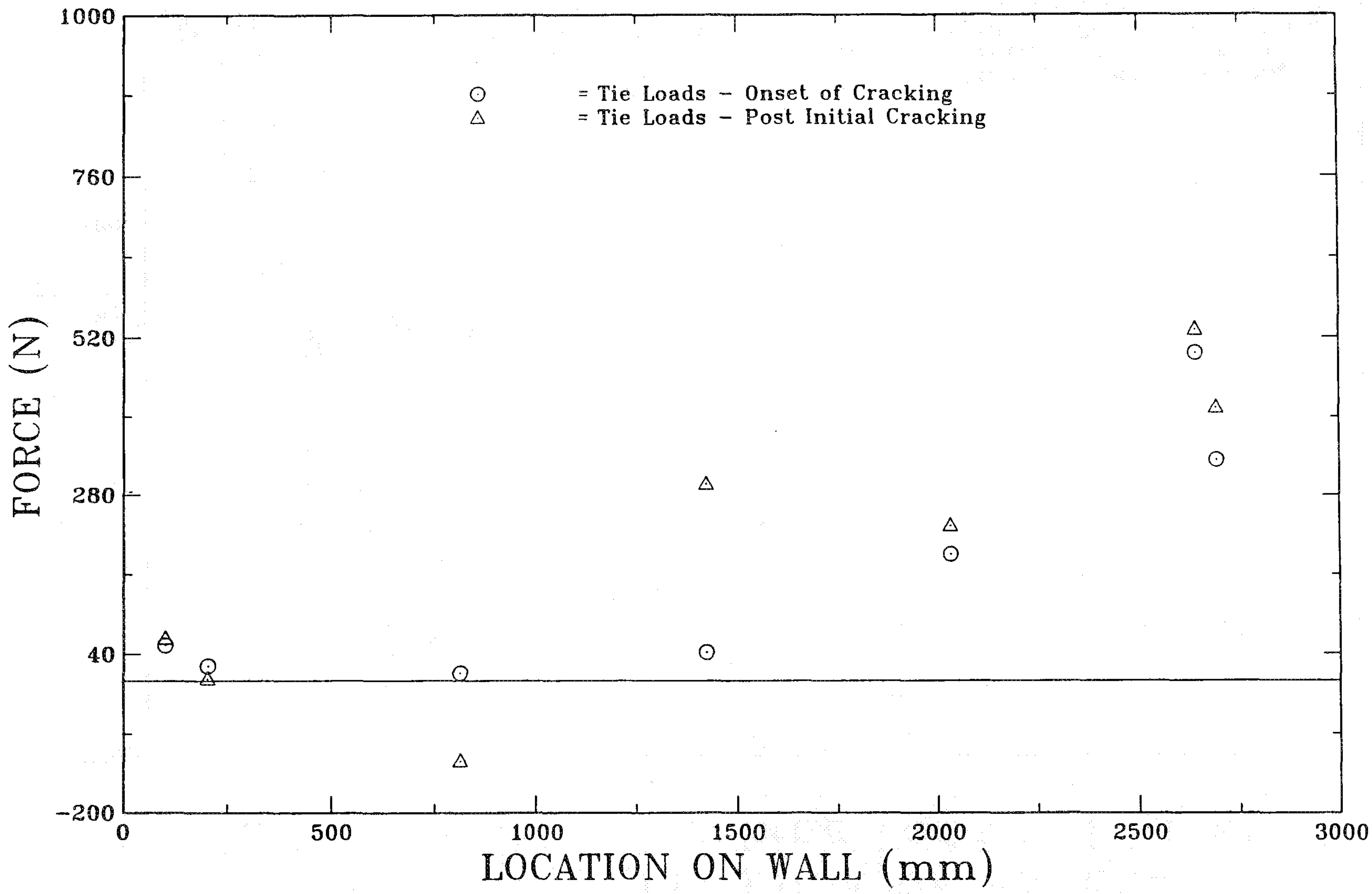



CASE G - SET TRACK STIFFNESS TO INFINITY
Middle Stud Cracking Load = 2.25 kPa
3000
2000
1000
0
LOCATION ON WALL
0
2
4
6
8
COMPONENT DISPLACEMENT (mm)
= Stud - Onset of Cracking
= Veneer - Onset of Cracking
= Stud - Post Initial Cracking
= Veneer - Post Initial Cracking

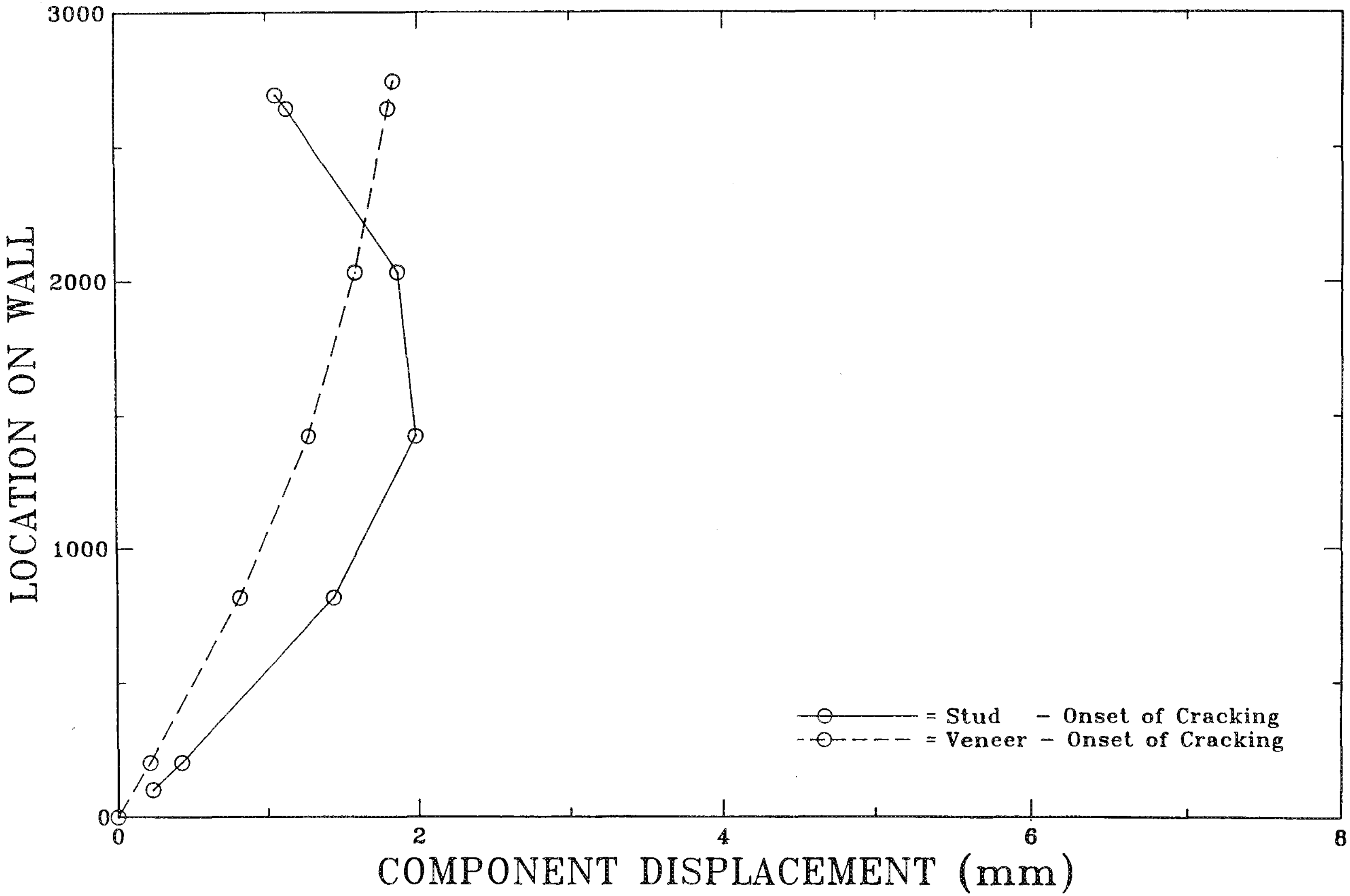
## CASE H – FIX EDGE OF VENEER
## Middle Stud Cracking Load = 1.81 kPa



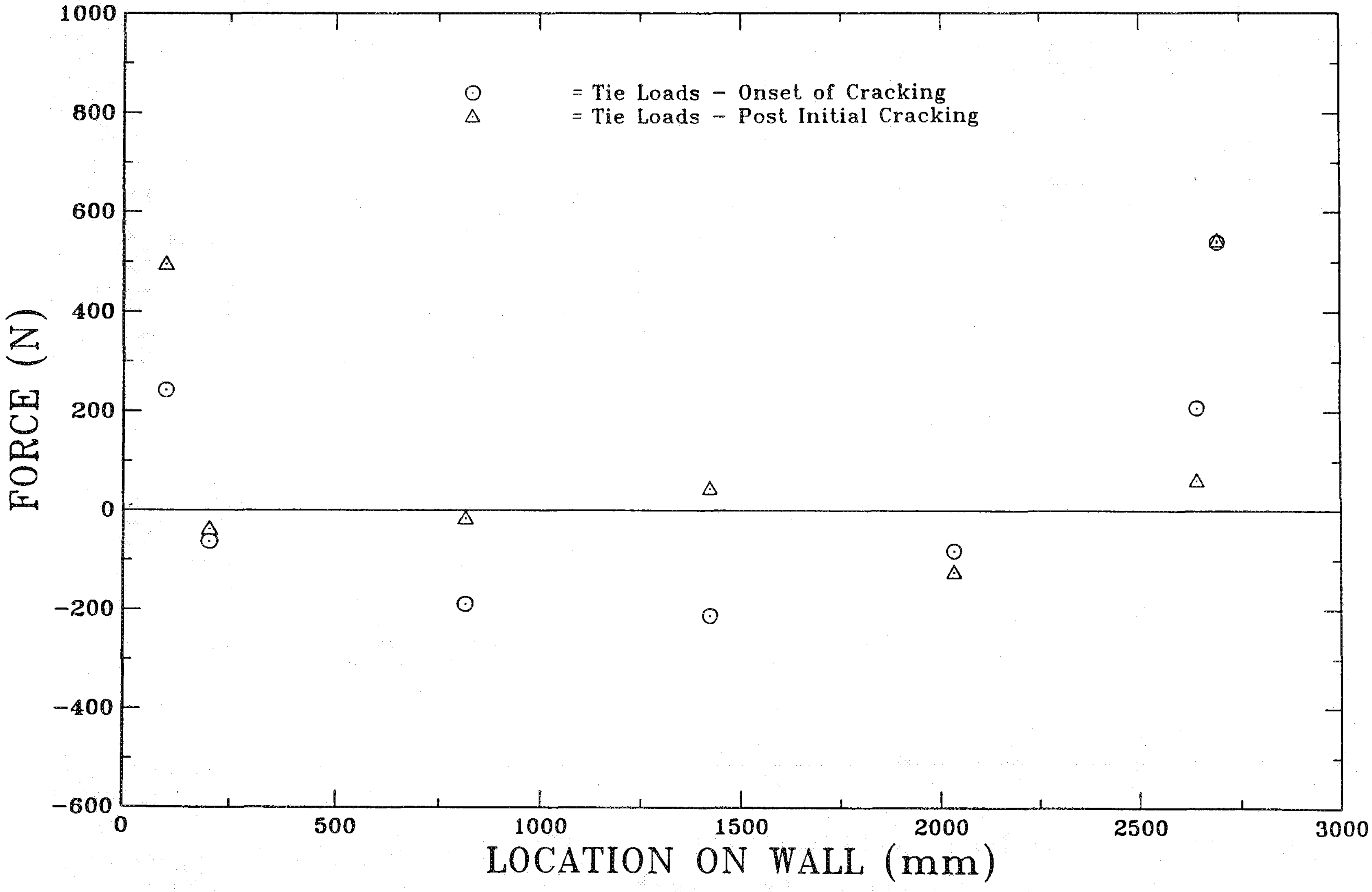
CASE 5H - TIE AND TRACK REACTIONS



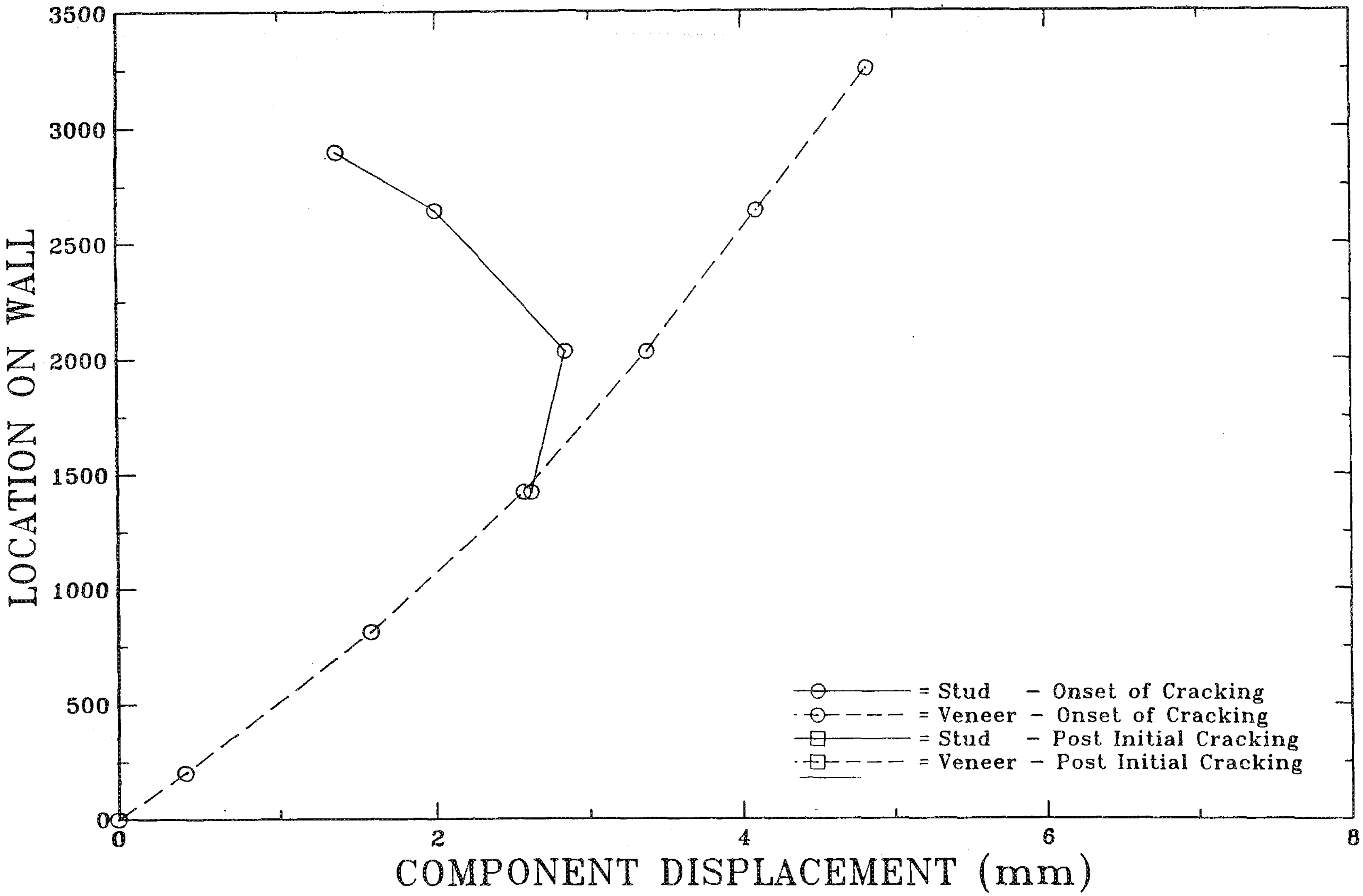
CASE J – Height of Veneer = 3,251 mm
Middle Stud Cracking Load = 1.09 kPa

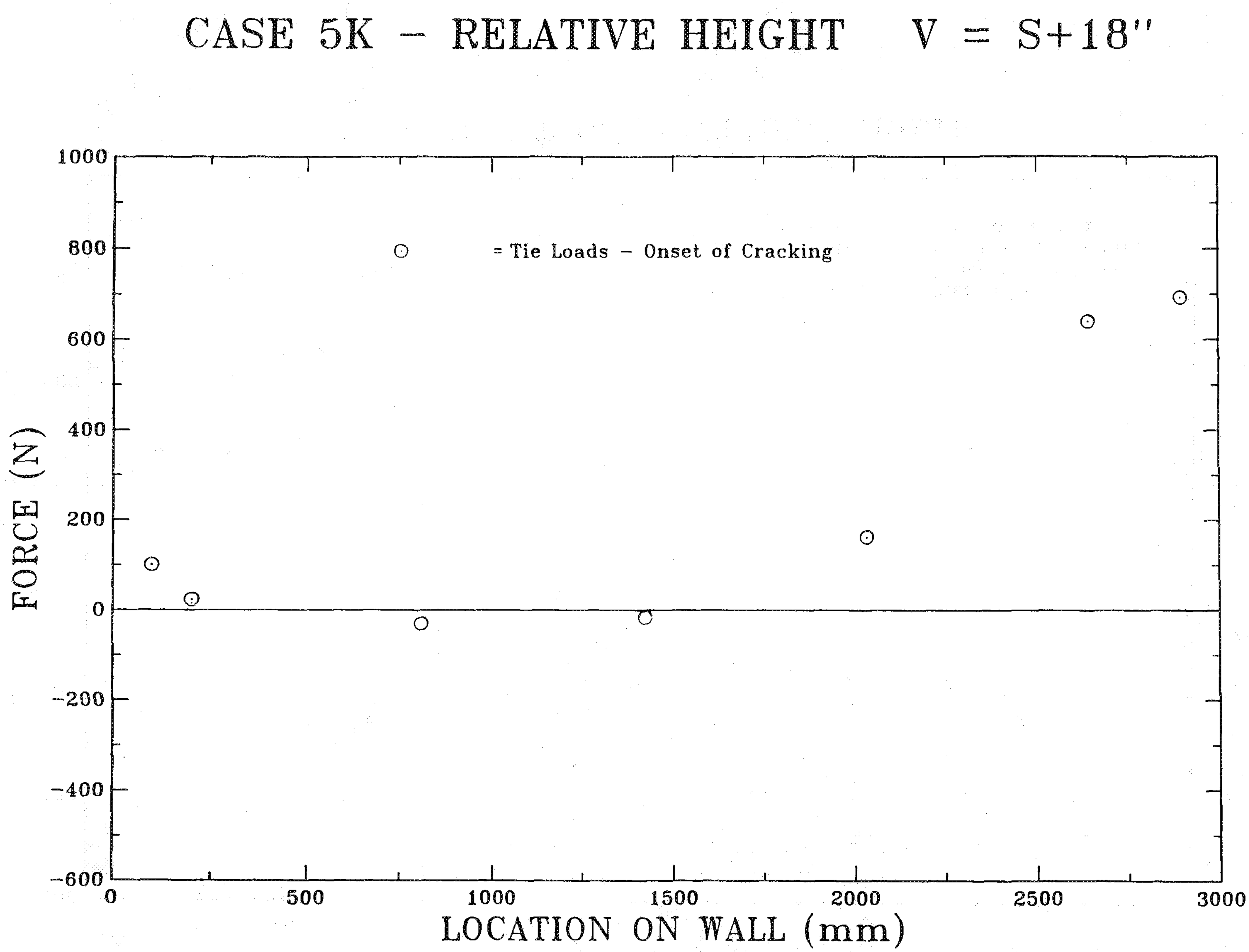


# CASE 5J - VENEER HEIGHT = 3251.2 mm

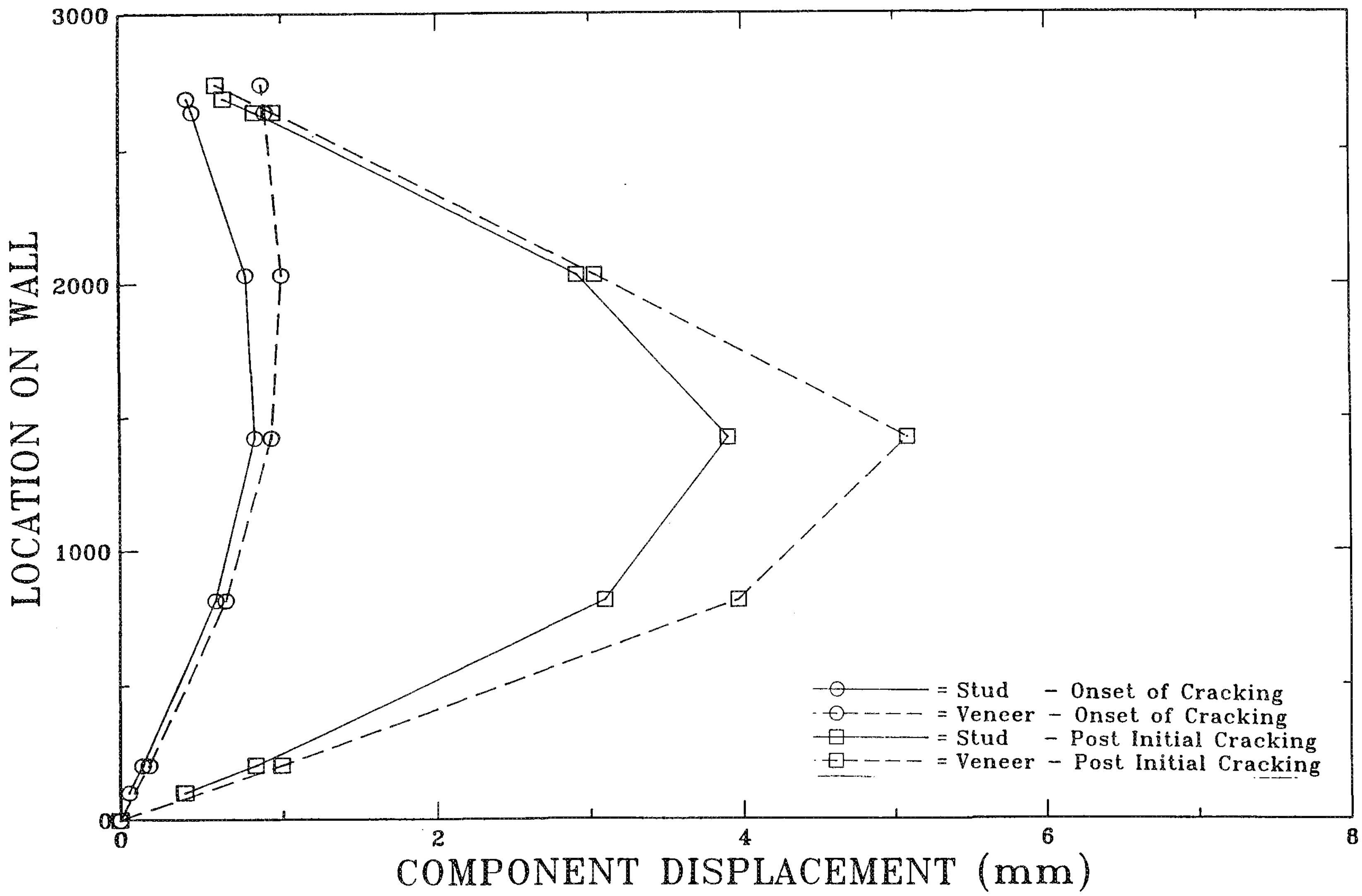


CASE K - Relative Height h=3.25 V=S+18"
Quarter Stud Cracking Load = 1.40 kPa

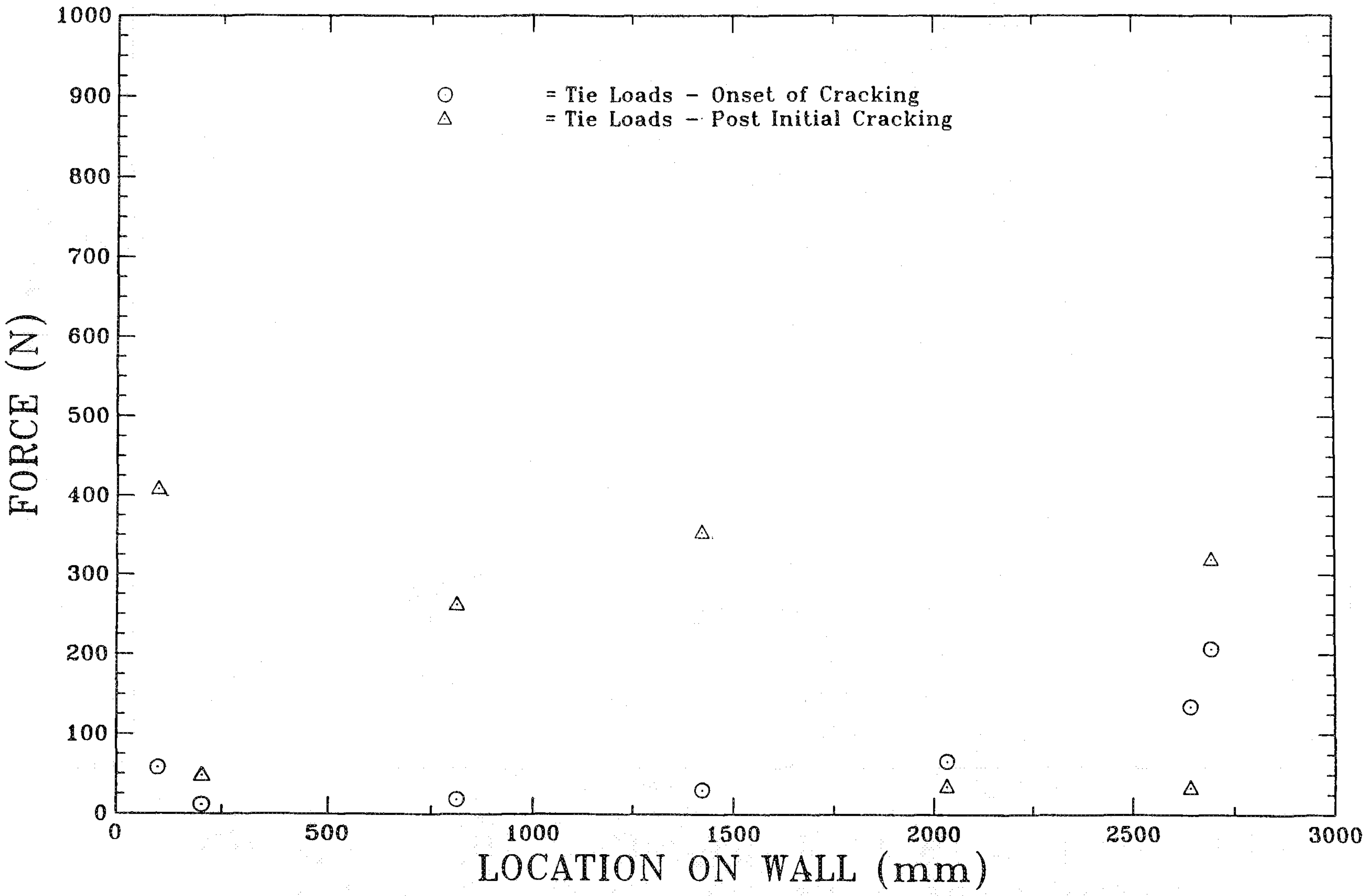



CASE 5K - RELATIVE HEIGHT V = S+18"
= Tie Loads - Onset of Cracking
FORCE (N)
LOCATION ON WALL (mm)
1000
800
600
400
200
0
-200
-400
-600
0
500
1000
1500
2000
2500
3000

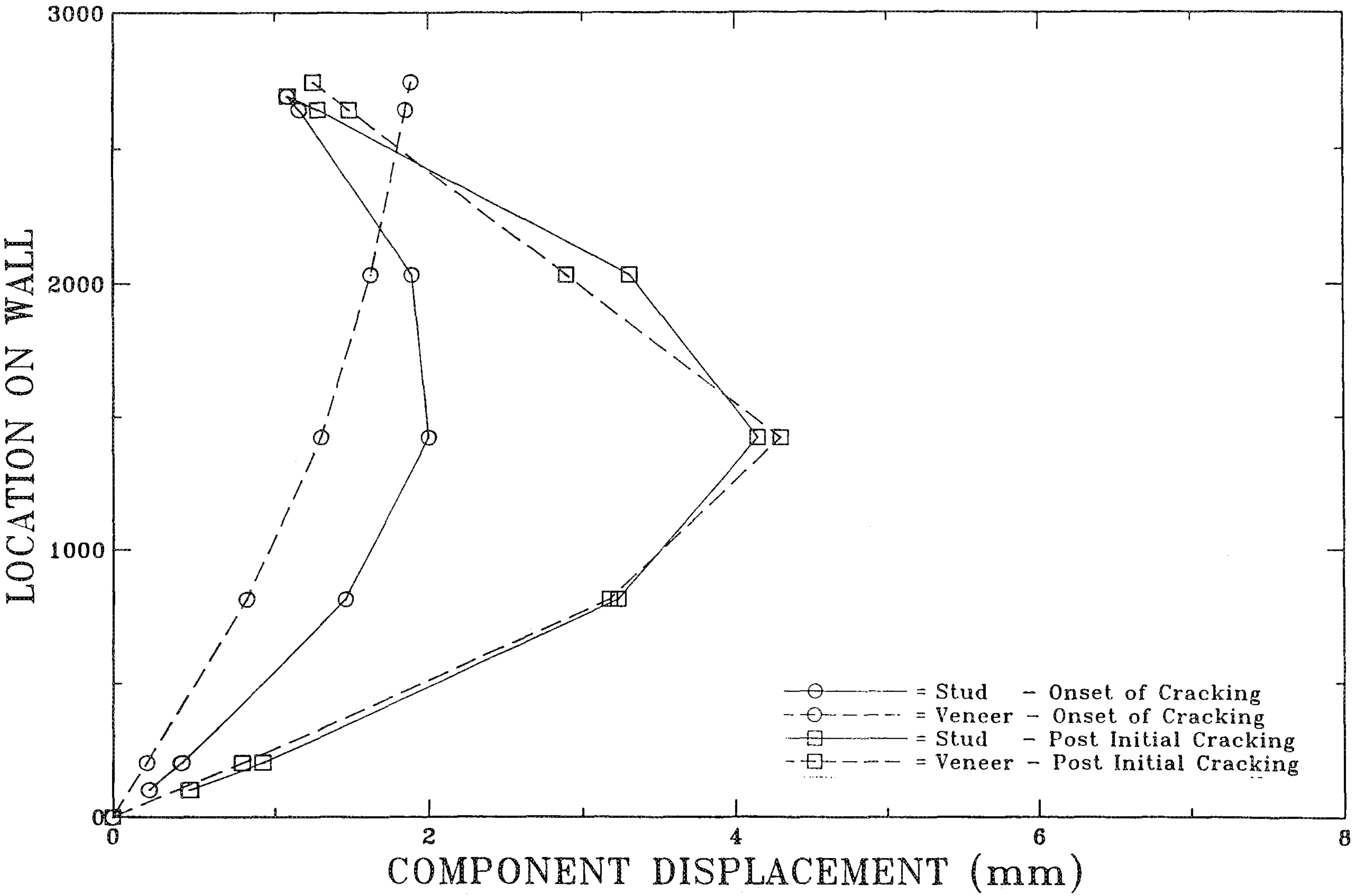
CASE L - TOP VENEER SUPPORTED
Middle Stud Cracking Load = 1.214 kPa

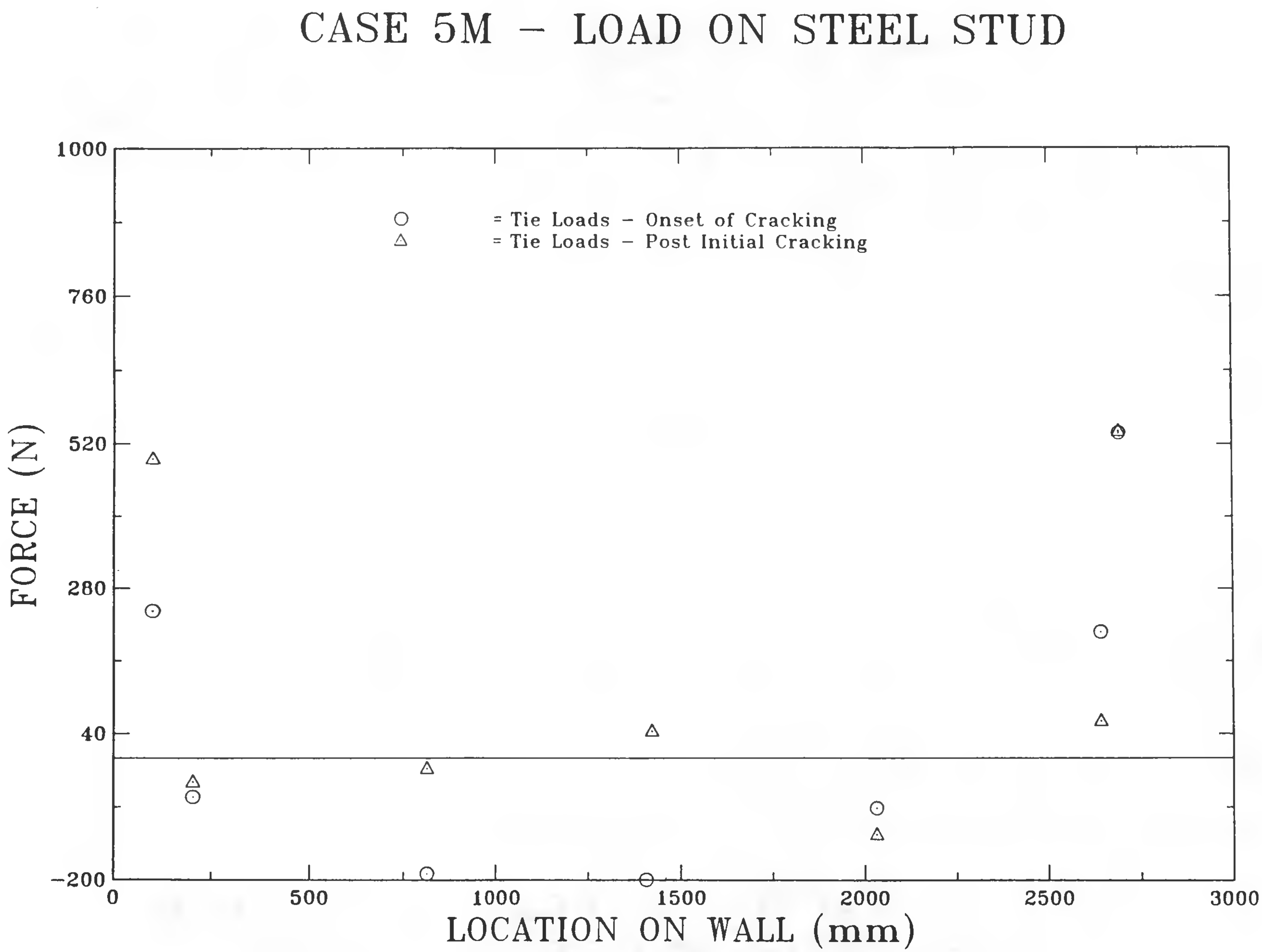


## CASE L – VENEER SUPPORTED



CASE M - LOAD ON STEEL STUD
Middle Stud Cracking Load = kPa





CASE 5M – LOAD ON STEEL STUD
= Tie Loads – Onset of Cracking
= Tie Loads – Post Initial Cracking
FORCE (N)
1000
760
520
280
40
-200
0
500
1000
1500
2000
2500
3000
LOCATION ON WALL (mm)