RESEARCH REPORT

Renovation Strategies for Brick Veneer Steel Stud Wall Construction: Task 2 Four Remedial Tie Systems for BV/SS Walls - Development & Conformance Testing

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RENOVATION STRATEGIES FOR BRICK VENEER STEEL STUD WALL CONSTRUCTION - TASK 2

Four Remedial Tie Systems For BV/SS Walls - Development & Conformance Testing

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Part IX

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Executive Summary

Introduction

Over the last 10 years, the performance of the clay brick veneer/steel stud (BV/SS) enclosure system, especially on multi-storey residential buildings, has received a great deal of attention. Many buildings have BV/SS enclosures and many of these have experienced or are experiencing problems. Not only is repair expensive, but there is also considerable uncertainty as to the level and extent of deterioration and damage, particularly the corrosion of metal components i.e., the ties, the stud system and the self-tapping screws.

It is difficult to decide on the form and extent of remedial action. The design professional faces a real dilemma when choosing an appropriate building repair strategy. If legal action is involved, there is considerable pressure to prescribe an overly conservative, "Cadillac" solution. On the other hand, there is the question of knowing what to do about those BV/SS walls that have yet to exhibit a visible problem but are known to be vulnerable and likely to experience problems. There have been numerous building investigations and attendant litigation. Large sums have been spent on R and D studies and field investigations. Studies have been directed at the structural requirements for the steel studs and the tie systems and at building science considerations such as air leakage, vapour transmission, resistance to heat flow and water penetration. The Canada Mortgage and Housing Corporation has been a prime mover in initiating much of the work in Canada.

A research and development contract was awarded to the Building Engineering Group (BEG) by CMHC to assess methods of repairing brick veneer/steel stud walls. The primary objective of this multi-task research project was to develop various strategies for the remediation and, thus, the control or avoidance of problems in existing BV/SS wall systems. Five tasks and related reports were identified :

- Task 1 : Brick Ties - Options for Remediation
- Task 2 : Four Remedial Tie Systems--Development and Conformance Testing
- Task 3 : Some Performance Considerations
- Task 4 : Dinal Remedial Tie System
- Task 5 : Summary Report

The main objective of the first task of the research study was to identify, demonstrate, assess, and document methods of providing supplemental ties on BV/SS buildings. Of particular interest were retrofit procedures that could be conducted from the interior.

Task 2 involved a test program to establish and document the performance of retrofit tie systems. Four retrofit tie systems were chosen; largely on the basis of the eleven retrofit tie systems evaluated in Task 1. The report that follows documents Task 2.

Objectives

The Task 2 project had the following objectives:

(i) to evaluate and assess the capabilities of four retrofit tie systems; two suited to retrofit from the exterior and two for installation working solely from the interior.

(ii) to identify and apply the relevant performance requirements to the four retrofit tie systems.

(iii) to discuss the various issues involved in connecting the brick veneer to the steel stud framing by means of ties, especially from a repair or retrofit point of view.

The performance of four proprietary retrofit tie systems has been assessed with respect to structural performance specifically structural safety and serviceability. Other performance considerations such as the effect on wall system air leakage, corrosion potential, thermal bridging, etc. are the subject of a separate study (Task 3).

Before starting the experimental work, a review of the results of recent R and D, relevant trade literature as well as new or impending standards was conducted in order to ensure that the appropriate performance requirements were properly identified and understood. The second chapter of the report also provides the context for evaluating the test results and assessing the merits of each retrofit tie system.

Program

The four retrofit tie systems chosen for detailed examination were the:

- **HE** Helifix Exterior Tie
- **DE** Dur-O-Wal Exterior Tie
- **HI** Helifix Interior Tie
- **DI** Dur-O-Wal Interior Tie

These ties are proprietary products. The two Helifix systems had never been used with steel stud framing. The two Dur-O-Wal systems are recent developments that have not had much use in either Canada or the United States. An extensive program of laboratory testing was conducted. Every attempt was made to ensure that all relevant aspects of performance were addressed and for this reason the test program was both extensive and comprehensive.

In setting up the test program it was realized that much of the previous testing on ties has limited relevance. Service load considerations such as the effect of cyclic loading, the magnitude of the initial stiffness, and the contribution of secondary displacements (the local deformation of the steel stud) needed to be quantified. Also safety considerations such as ductility and structural integrity needed to be taken into account.

In total 435 test of individual tie-stud connections were made. Each tie was connected to a length of stud, which, in turn, spanned either 65mm (151 "isolation" tests) or 400mm (284 "beam" tests) between the centre-line of supports. The "beam" tests permit an assessment of the influence of the deformation of the stud, both flange rotation and stud deflection. In many tests a preconditioning regime of 1000 cycles of 0.15 Kn (33Lbf) tension (pullout) and 0.15 Kn compression (push-in) at 1 Hz was applied. Each test was stopped only when either a significant and permanent loss of capacity had occurred or displacement was excessive (in excess of 15mm). 16, 18, 20, and 21 gauge steel studs were tested.

Outcome

Of the four retrofit systems tested it was evident that only one system, the Dur-O-Wal Exterior fix (using a Dur-O-Wal 1/4" dia. lagbolt into the steel stud and an expansion anchor to the brick) consistently developed sufficient strength, with satisfactory initial stiffness and acceptable displacements, to be used with all the gauges of steel stud framing tested, i.e., 16, 18, 20, and 21 gauge.

A summary of the pertinent design characteristics for all four tie systems, suitable for use by a designer, has been developed. The gauge of the stud is an important parameter. While 22, 24 or even thinner gauge studs may have been utilized in many buildings, it is recommended that caution should be excersized in any attempt to extrapolate the Task 2 test results to these thinner steel stud systems.

In particular it should be emphasized that for the:

- Dur-O-Wal Exterior Fix (lagbolt and expansion anchor) --- This tie system may be used with 16, 18, and 20 gauge steel studs.
- Helifix Exterior Fix (HRT80 dry fix in SS, polyester resin in BV) --- This retrofit fix is not recommended except, perhaps, for use with 16 or thicker steel stud. This tie system is the least stiff of the four systems tested.
- Helifix Interior Fix (HRT80 Tie Dry Fix) --- This tie system is suitable for use with 16 and, perhaps, 18 gauge steel stud.
- Dur-O-Wal Interior Fix (Stainless Steel Rod and Sleeve with Epoxy) --- This tie system could be used with 16, 18, 20 and even 21 gauge steel stud but the capacity of the tie is, in all cases, relatively low.

These recommendations have been summarized in a single Table which provides a simple means of choosing the appropriate supplementary tie system. Two other considerations that are important in design are the stiffness of back-up framing and whether to work from the interior or the exterior.

To demonstrate the practical relevance of the design recommendations for the tie-SS connection for buildings of various heights, different locations and varying tributary tie areas, various design charts have been developed. Provision has also been made for various "equivalent" tributary areas in order to account for the use of a flexible back-up.

An important consideration is whether to use an interior fix or an exterior fix. Largely because the interior fixes tested had to penetrate both flanges the interior fixes tend to have rather different, i.e., stiffer and potentially stronger, from a comparable exterior fix. In general, the ties in an interior fix are stiffer due to the attachment of the tie to both flanges. This reduces the amount of flange rotation and gives a stiffer connection. For instance, the Helifix Interior tie had less cyclic load displacement, less displacement at 0.45 kN and higher initial stiffness than the Helifix Exterior Tie.

Finally it needs to be stated that practical and effective methods to remediate the tie-stud connection in existing BV/SS walls do exist. In this project only four possible alternatives were investigated. Clearly further developmental work is required, especially in better implementation of the Helifix Tie. This report does provide a basis for the assessment of other tie remediation methods or improvements to the four methods examined.

Résumé

Introduction

Depuis dix ans, la performance des systèmes de mur à ossature d'acier et placage en brique d'argile, destinés spécialement aux bâtiments résidentiels collectifs, accapare beaucoup d'attention. De nombreux bâtiments en comportent et ils sont également très nombreux à avoir connu ou à connaître des problèmes. Non seulement les réparations coûtent-elles cher, mais on ne peut établir avec certitude l'ampleur de la détérioration et des dommages, en particulier de la corrosion des composants métalliques, à savoir les attaches, l'ossature métallique et les vis autotaraudeuses.

Il est difficile de décider du genre et de l'ampleur des mesures de réhabilitation. L'expert concepteur fait face à un véritable dilemme lorsque vient le temps de choisir une stratégie de réhabilitation tout indiquée pour le bâtiment. En cas de poursuite, des pressions considérables sont exercées pour que soit prescrite une solution par trop prudente. Par contre, que faire au sujet des murs à ossature d'acier et placage de brique qui n'affichent pas encore d'anomalie visible mais qui passent pour vulnérables et susceptibles d'occasionner des problèmes? Beaucoup de bâtiments ont été l'objet d'investigations et de litiges correspondants. De fortes sommes ont été consacrées à des activités de recherche et de développement ainsi qu'à des investigations sur le terrain. Des études ont porté sur les exigences structurales des poteaux d'acier et des attaches ainsi que sur les aspects de la science du bâtiment touchant notamment à l'étanchéité à l'air, à la transmission de vapeur, à la résistance au mouvement de chaleur et à l'infiltration d'eau. La Société canadienne d'hypothèques et de logement a été l'un des principaux organismes à amorcer la plupart des travaux menés au Canada.

La SCHL a adjugé au Building Engineering Group (BEG) un contrat de recherche et de développement axé sur l'évaluation de méthodes de réhabilitation de murs à ossature d'acier et placage de brique. L'objectif premier de cette recherche multitâche consistait à mettre au point différentes stratégies de réhabilitation et, par conséquent, à contrôler ou à éviter la manifestation de problèmes dans de tels murs. Cinq tâches devant donner lieu à des rapports connexes ont été désignées :

- Tâche 1 : Attaches de la brique - Options de réhabilitation
- Tâche 2 : Quatre systèmes d'attaches - Élaboration et essais de conformité
- Tâche 3 : Aspects de la performance
- Tâche 4 : Système d'attache Dinal
- Tâche 5 : Rapport sommaire

La Tâche 1 poursuivait l'objectif principal de désigner, de démontrer, d'évaluer et de documenter des façons de pourvoir d'attaches supplémentaires les bâtiments à ossature d'acier et placage de brique. Elle privilégiait les techniques de pose depuis l'intérieur.

La Tâche 2 comportait un programme d'essais destiné à établir et à documenter la performance des systèmes d'attaches de consolidation. Quatre systèmes d'attaches ont été choisis, en grande partie en fonction des onze évalués dans le cadre de la Tâche 1. Le rapport qui suit documente la Tâche 2.

Objectifs

Voici les objectifs de la Tâche 2 :

i) évaluer la capacité de quatre systèmes d'attaches, dont deux se se posent de l'extérieur et deux uniquement de l'intérieur.
ii) établir et appliquer les exigences de performance correspondantes aux quatre systèmes d'attaches.
iii) traiter des différents aspects concernant le raccordement du placage de brique à l'ossature d'acier au moyen d'attaches, spécialement du point de vue de la réparation ou de la réhabilitation.

Quatre systèmes d'attaches de marque déposée ont été évalués en fonction de leur performance structurale sur le plan de la sécurité et de la fonctionnalité. L'effet sur l'étanchéité à l'air du mur, les possibilités de corrosion, les ponts thermiques, etc., font l'objet d'une étude distincte (Tâche 3).

Avant d'entreprendre les travaux expérimentaux, il a fallu revoir les résultats des récents travaux de recherche et de développement, la documentation spécialisée correspondante de même que les normes nouvelles ou imminentes pour s'assurer de bien identifier et saisir les exigences de performance appropriées. Le deuxième chapitre du rapport livre également le contexte pour évaluer les résultats d'essais et les mérites de chaque système d'attache.

Programme

Voici les quatre systèmes d'attaches choisis en vue d'un examen approfondi :

- l'attache extérieure Helifix **HE**
- l'attache extérieure Dur-O-Wal **DE**
- l'attache intérieure Helifix **HI**
- l'attache intérieure Dur-O-Wal **DI**

Il s'agit de produits de marque déposée. Les deux systèmes Helifix n'ont jamais été utilisés avec une ossature d'acier. Les deux systèmes Dur-O-Wal, de confection récente, n'ont pas encore eu d'usage vraiment répandu au Canada ou aux États-Unis. Un programme étendu d'essais en laboratoire a été mené. Tout a été mis en oeuvre pour que tous les aspects correspondants de la performance soient traités et c'est pourquoi le programme d'essais a été à la fois étendu et complet.

Au moment de mettre sur pied le programme d'essais, on s'est rendu compte que la plupart des essais antérieurs ayant porté sur les attaches avaient peu de pertinence. L'effet des surcharges cycliques, l'importance de la rigidité d'origine et les déplacements secondaires (déformation locale des poteaux d'acier) devaient être quantifiés, tout comme il fallait tenir compte de considérations de sécurité telles la ductilité et la solidité structurale.

En tout, 435 essais de raccordements attaches-poteaux ont été effectués. Chaque attache était raccordée à un poteau dont la portée entre l'axe des supports correspondait à 65 mm (151 essais individuels) ou à 400 mm (284 essais groupés). Les essais groupés permettaient d'évaluer l'influence de la déformation des poteaux, ainsi que la rotation des ailes et le fléchissement des poteaux. Dans de nombreux tests, un régime de préconditionnement de 1 000 cycles de 0,15 kN (33 lb/pi) en tension (arrachement) et de 0,15 kN en compression (enfoncement) à 1 Hz a été appliqué. Chaque essai ne prenait fin que lorsqu'il se produisait une perte de capacité appréciable permanente ou un déplacement excessif (supérieur à 15 mm). Les essais ont porté sur des poteaux d'acier d'épaisseur 16, 18, 20 et 21.

Résultats

Parmi les quatre systèmes d'attaches testés, il était évident que seul le système d'attache Dur-O-Wal se posant de l'extérieur (avec tire-fonds Dur-O-Wal de 1/4 po de diamètre s'enfonçant dans le poteau d'acier et coquille d'expansion dans la brique) donnait toujours une résistance suffisante, de même qu'une rigidité initiale satisfaisante et un déplacement acceptable, pour toutes les épaisseurs de poteaux d'acier testés, soit 16, 18, 20 et 21.

Un résumé des caractéristiques conceptuelles pertinentes des quatre systèmes d'attaches a été établi pour fins d'emploi par un concepteur. L'épaisseur du poteau revêt de l'importance. Bien que des poteaux d'épaisseur 22 ou 24 ou même plus minces aient été utilisés dans de nombreux bâtiments, il est recommandé d'user de prudence dans toute tentative d'extrapoler les résultats d'essai de la Tâche 2 pour les systèmes à poteaux plus minces.

Il faut souligner, en particulier, que :

- le système extérieur Dur-O-Wal (tire-fonds et coquille d'expansion) peut s'utiliser avec des poteaux d'acier d'épaisseur 16, 18 et 20.

- l'usage du système extérieur Helifix (HRT80, pose à sec dans les poteaux d'acier, résine de polyester dans le placage de brique) n'est pas recommandé, sauf peut-être avec des poteaux d'acier d'épaisseur 16 ou supérieure. Ce système d'attache s'avère le moins rigide des quatre testés.

- le système intérieur Helifix (pose à sec de l'attache HRT80) convient aux poteaux d'acier d'épaisseur 16 et peut-être 18.

- le système intérieur Dur-O-Wal (tige d'acier inoxydable et manchon avec époxy) pourrait s'employer avec des poteaux d'acier d'épaisseur 16, 18, 20 et 21, mais la capacité de l'attache est dans tous les cas relativement faible.

Résumées dans un seul tableau, ces recommandations fournissent un moyen simple de choisir le système d'attache supplémentaire approprié. La rigidité du mur de fond et la possibilité de réaliser les travaux de l'intérieur ou de l'extérieur constituent deux autres importantes considérations.
Pour illustrer la pertinence pratique des recommandations conceptuelles touchant le raccordement des attaches et poteaux d'acier de bâtiments de différentes hauteurs, selon différents endroits et diverses surfaces d'attaches tributaires, divers tableaux conceptuels ont été établis. On a prévu diverses surfaces tributaires «équivalentes» dans le but de tenir compte du recours à un mur de fond flexible.

La réalisation des travaux depuis l'intérieur ou l'extérieur revêt un aspect important. Surtout parce que les attaches intérieures testées devaient pénétrer les deux ailes, les attaches posés de l'intérieur affichaient généralement un comportement plutôt différent, se révélant plus rigides et possiblement plus résistantes, comparativement aux attaches extérieures semblables. En général, les attaches se posant de l'intérieur étaient plus rigides puisqu'elles étaient raccordées aux deux ailes des poteaux, ce qui réduit le risque de rotation des ailes et ajoute à la rigidité du raccordement. Par exemple, l'attache intérieure Helifix subissait moins de déplacement sous l'effet de surcharges cycliques, moins de déplacement sous une charge de 0,45 kN et avait une rigidité d'origine supérieure à celle de l'attache extérieure Helifix.

Enfin, il faut faire ressortir qu'il existe des méthodes pratiques et efficaces de consolider le raccordement attaches-poteaux des murs existants à ossature d'acier et placage de brique. Cette recherche n'a porté que sur quatre possibilités. D'autres travaux de développement s'imposent de toute évidence, surtout en ce qui concerne l'amélioration de la mise en oeuvre des attaches Helifix. Le présent rapport jette les bases pour l'évaluation d'autres méthodes de consolidation des attaches ou l'amélioration des quatre méthodes examinées.

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Acknowledgements

This study is the second task in an extensive, five task R&D project investigating strategies for the remediation of Brick Veneer/Steel Stud Wall Systems for Canada Mortgage and Housing Corporation. We would like to thank Mr. Jacques Rousseau for initiating and managing this unique project; unique in that it required the collaboration of the government, a university, a number of consultants and numerous companies involved in the brick masonry business.

We would like to thank the tie manufacturers/distributors that took part in both Tasks 1 and 2 of these projects namely; Mr. Ken Banks and, particularly, Mr. Ken Crooks of Dur-O-Wal Ltd. and Mr. Pat Sweeney of Blok-Lok Ltd.

We greatly appreciate the assistance of Mr. Leo Hansen, Dick Powers and Ralph Korchensky, technicians in the Civil Engineering Department of the University of Waterloo, for their assistance with the experimental work.

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1. INTRODUCTION

1.1 Background

Over the last 10 years, the performance of the clay brick veneer/steel stud (BV/SS) enclosure system, especially on multi-storey residential buildings has received a great deal of attention. Many buildings have BV/SS enclosures and many of these have experienced or are experiencing problems. Not only is repair expensive, but there is also considerable uncertainty as to the level and extent of deterioration and damage, especially the corrosion of metal components i.e., the ties, the stud system and the self-tapping screws. It is particularly difficult to decide on the form and extent of remedial action. The design professional faces a real dilemma when choosing an appropriate building repair strategy. If legal action is involved, there is considerable pressure to prescribe an overly conservative, "Cadillac" solution. On the other hand, there is the question of knowing what to do about those BV/SS walls that have yet to exhibit a visible problem but are known to be vulnerable and likely to experience problems. There have been numerous building investigations and attendant litigation. Large sums have been spent on R and D studies and field investigations. Studies have been directed at the structural requirements for the steel studs and the tie systems and at building science considerations such as air leakage, vapour transmission, resistance to heat flow and water penetration. The Canada Mortgage and Housing Corporation has been a prime mover in initiating much of the work in Canada.

Recently a research and development project was awarded to the Building Engineering Group (BEG) by CMHC to assess methods of repairing brick veneer/steel stud walls. The primary objective of this multi-task research project was to develop various strategies for the remediation and, thus, the control or avoidance of problems in existing BV/SS wall systems. Five tasks were identified :
Task 1 : Brick Ties - Options for Remediation
Task 2 : Four Remedial Tie Systems--Development and Conformance Testing
Task 3 : Some Performance Considerations
Task 4 : Dinal Remedial Tie System
Task 5 : Summary Report

A report that documents the work conducted in Task 1, the initial study, has been completed. The main objective of the first task of the research study was to identify, demonstrate, assess, and document methods of providing supplemental ties on BV/SS buildings. Of particular interest were retrofit procedures that could be conducted from the interior.

Task 2 involved a test program to establish and document the performance of retrofit tie systems. Four retrofit tie systems were chosen; largely on the basis of the eleven retrofit tie systems evaluated in Task 1. This report documents Task 2.

1.2 Objectives

This project (Task 2) had the following objectives:

(i) to evaluate and assess the capabilities of four retrofit tie systems; two suited to retrofit from the exterior and two for installation working solely from the interior.

(ii) to identify and apply the relevant performance requirements to the four retrofit tie systems.

(iii) to discuss the various issues involved in connecting the brick veneer to the steel stud framing by means of ties, especially from a repair or retrofit point of view.

1.3 Approach and Scope

The four retrofit tie systems chosen for detailed examination are:

- HE Helifix Exterior Tie
- DE Dur-O-Wal Exterior Tie
- HI Helifix Interior Tie
- DI Dur-O-Wal Interior Tie

These ties are proprietary products. The two Helifix systems have never been used with steel stud framing and, in this regard, the work that follows is developmental. The two Dur-O-Wal systems are recent developments that have not had much use in either Canada or the United States. An extensive program of laboratory testing was conducted. Every attempt has been made to ensure that all relevant aspects of performance were addressed and for this reason the test program was both extensive and comprehensive. In particular we have been guided by, but not restricted by, the provisions of two recently revised masonry standards namely:

- CSA Standard A370-94, Connectors for Masonry[1]
- CSA Standard S304.1-94, Masonry Design for Buildings[2]

At the time of carrying out this project the new Standards had yet to be issued and we were guided by the various drafts and some Committee correspondence.

In setting up the test program it was realized that much of the previous testing on ties has limited relevance. In particular service load considerations such as the effect of cyclic loading, the magnitude of the initial stiffness, and the contribution of secondary displacements (the local deformation of the steel stud) were not quantified. Moreover safety considerations such as ductility and structural integrity needed to be taken into account.

Before starting the experimental work it was necessary to review recent R and D, the trade literature as well as new or impending standards in order to ensure that the relevant performance requirements were properly identified and understood. In order to demonstrate the context within which tie systems are likely to be selected and future BV/SS wall systems will be designed, Chapter 2 of this report presents, in some detail, some background, review and analysis of the issues involved. Chapter 2 also provides the context for evaluating the test results and assessing the merits of each retrofit tie system.

Chapters 3 and 4 and the Appendices document the test program and the test results. In Chapter 5 the test results are discussed, compared with other tie systems and evaluated with particular emphasis on considerations of structural serviceability and safety. The scope of this task is largely limited to the structural performance of BV/SS wall systems. Although much of what is reported applies to BV and other backup systems, specifically concrete block, this study only addresses BV/SS systems. Non-structural considerations for these retrofit tie systems such as the potential for air leakage, corrosion, etc. are the subject of another project, Task 3, and another report. Chapter 6 documents both conclusions and recommendations. A comprehensive executive summary has been included to provide the non-specialist an outline of what was done and a synthesis of the outcomes.

References

1 CSA Standard A370-94, Connectors for Masonry, Canadian Standards Association, Rexdale, Ontario, 1994

2 CSA Standard S304.1-94, Masonry Design for Buildings, Canadian Standards Association, Rexdale, Ontario, 1994

2. BRICK VENEER/STEEL STUD (BV/SS) WALL SYSTEMS

2.1 General Description of the BV/SS Wall System

The brick veneer/steel stud wall system is typically used as exterior infill, spanning one and sometimes two floors in multistorey buildings. In order to fulfill its primary role as an environmental separator, proper design and installation of the numerous components of this infill wall are essential. The following section provides a brief description of a BV/SS wall system. The intent is to set the context for a more detailed discussion of the structural characteristics and performance of BV/SS wall systems. A more detailed description of the various components of a brick veneer/steel stud system is contained in many other documents.[1,2,3,4]

Figure 2-1 shows a typical cross-section of a BV/SS wall system. The wall consists of multiple layers all of which are important to the overall performance of the wall system. The brick veneer is the first line of defense against the external environment. Contributing to the control of water penetration is one of the most important functions of the brick veneer. The ability of the brickwork to resist water penetration depends on the quality of the materials and, most importantly, on the quality of workmanship used to build the wall. Of particular importance are the following :

1) full, well tooled mortar joints
2) good contact and adhesion between the mortar and brick
3) properly caulked joints at intersections with other wall components

The cavity behind the brick veneer is typically or should be 25mm to 50mm (1" to 2") in width. Mortar droppings, dams, and bridges in the cavity are not desirable as they will provide paths for water to pass across the cavity. Mortar bridges will also cause some structural interaction between the brick veneer and the steel stud backup.

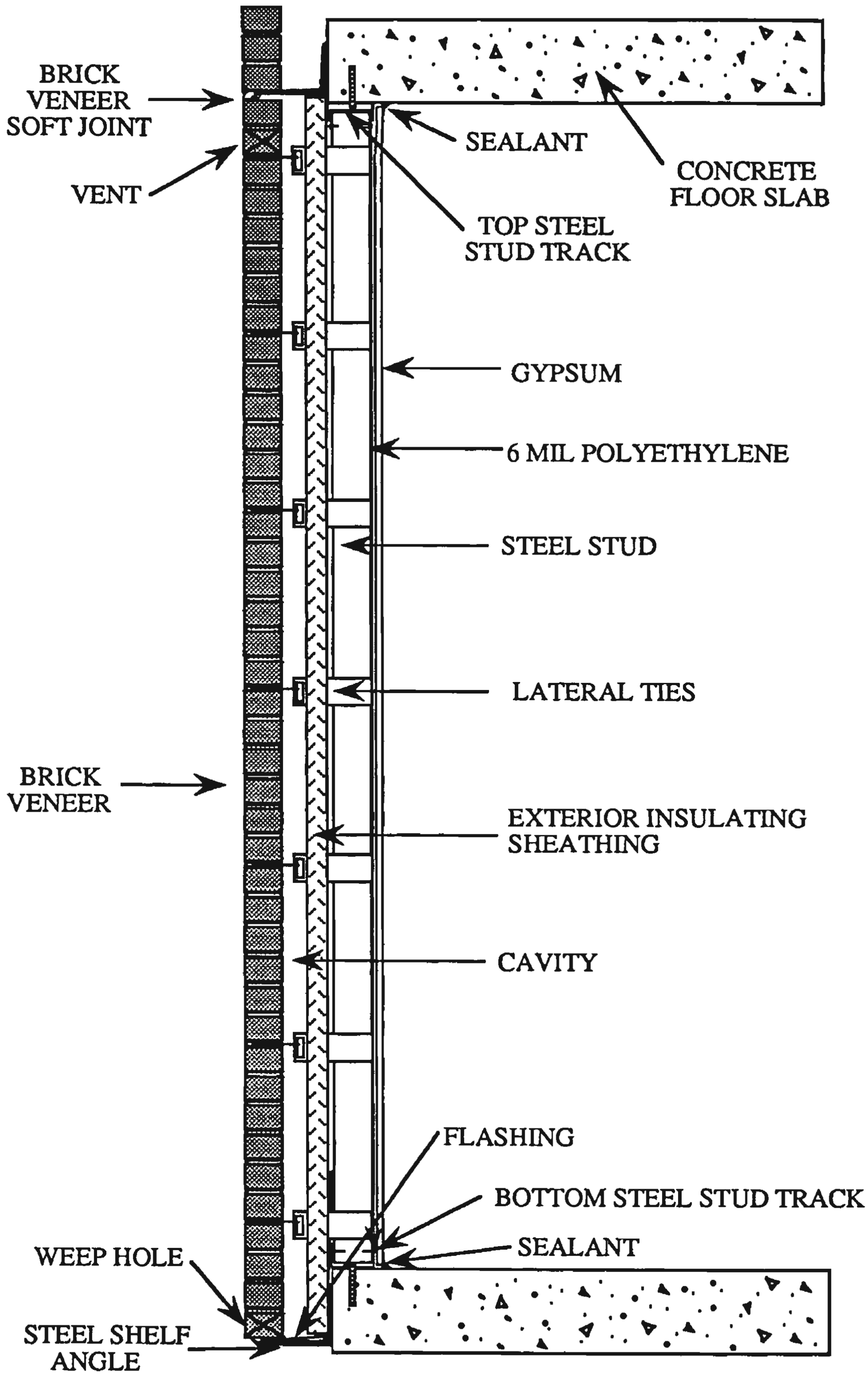


Figure 2-1 --- Cross Section of Representative BV/SS Wall System

BV/SS wall systems can be insulated in different ways. Formally the favoured approach was to provide fiberglass batt insulation in the stud space and use a non-insulating exterior sheathing such as gypsum board. Currently, designers are using an insulating exterior sheathing varying in thickness from 25mm to 75mm (1" to 3") with or without fibreglass batt insulation in between the steel studs. Two dimensional, steady state, thermal gradient analyses have shown that up to 75mm of insulated exterior sheathing with no batt insulation may be the preferred strategy to ensure that, under extreme winter conditions, a dew point does not exist in the steel stud section.[5] A dew point within the stud space is not desirable if there is any moist exfiltrating air that can condense within the backup system, thus greatly increasing the potential for corrosion. The structural benefits of exterior sheathing depend upon the properties of the sheathing. Most exterior sheathings are not very stiff in the out-of-plane direction and thus will not act compositely with the steel studs. However, some sheathings are stiff enough to provide some restraint to the distortion of the flanges of the steel studs. Reduced thermal bridging is another benefit from using exterior insulating sheathing which, in turn, reduces the liklihood of "telegraphing" or "dusting" of the stud on the drywall.

Figure 2-2 shows the exterior elevation of the wall system. The brick masonry is supported at each floor level typically by shelf angles and in some instances by the floor slab. The shelf angles are hidden from view by the caulking that seals the "soft" horizontal joint below the shelf angle. This control joint is important in isolating the masonry from the upper floor to avoid stresses due to differential movement due to deflection of the floor slabs or the spandrel beams, moisture expansion of the clay brick veneer, thermal movement, etc. Mainly because of the moisture and thermally induced movement of the clay brick veneer, the brick veneer is also separated into horizontal sections by means of vertical control joints. A good design will have vertical control joints at least every 12m and at discontinuities in the brick veneer such as windows, corners and reveals.[6,7,8]

Also noticeable from the exterior are the weep and vent holes. These are typically spaced every 600mm. Weep holes located in the bottom course above the flashing enable water within the wall system to drain out. Provided these holes permit air flow they are also air vents. Vent holes located at the top of the wall are often provided in BV/SS walls. These upper vent holes plus the lower vent holes allow air flow in the air space behind the brick

Figure 2-2 --- Exterior Elevation of BV/SS Wall System

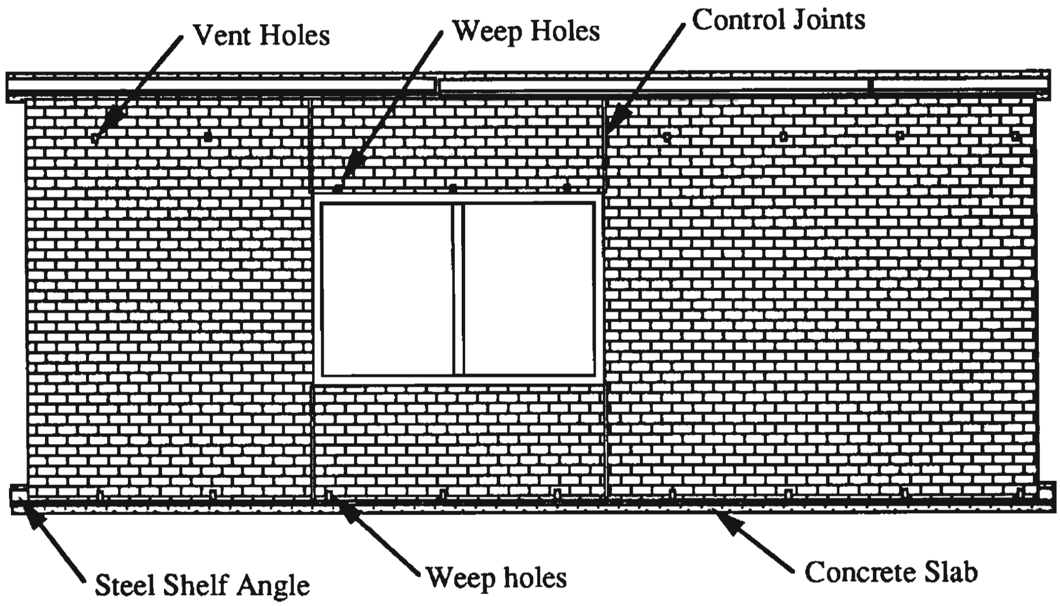
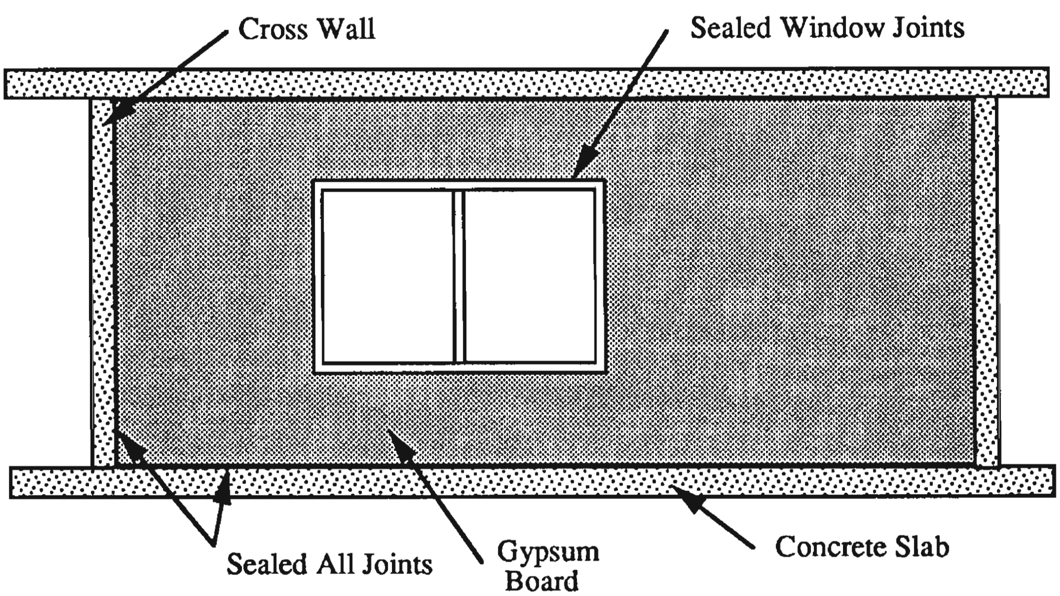


Figure 2-3 --- Interior Elevation of BV/SS Wall System



veneer and permit a degree of convective drying. However there is some controversy about the need for top venting and the net effectiveneess of any convective air flow.

In the majority of infill wall systems openings exist in the form of windows or sliding doors. These discontinuities require special attention to be paid to the design and installation of the steel stud backup and the lateral ties.

Figure 2-3 is an interior elevation of the interior of a typical wall system. The interior drywall spans over the steel studs. The plane of the drywall is often used as the air barrier in these wall systems. Proper sealing of the intersection of the drywall and the side walls, roof, and the floor is essential to provide a continuous air barrier. All window joints, electrical boxes and other penetrations must also be air sealed.

Figure 2-4 shows the exterior elevation with the brick veneer removed. There are numerous joints in the exterior sheathing. If the plane of the exterior sheathing is to be used as the air barrier, the sealing of these joints is very important. Because the air seal should last the lifetime of the wall system, repair or resealing of the air barrier at this plane would be prohibitively expensive. Typically the joint contribution of the interior gypsum board, polyethylene vapour retarder and perimeter sealing is used to provide the primary air barrier. However, the exterior sheathing should also be sealed, primarily against water. This exterior sheathing must be vapour permeable to allow moist air in the stud space to migrate out to the cavity and, subsequently, to the exterior.

The flashing at the bottom of the wall redirects the water that enters the cavity to the exterior through the weep holes. Flashing is made from materials such as sheet metal, laminated foil, vinyl, bituminous membranes,etc. The detailing and installation of the flashing are very important as this is the critical feature of the drainage system, as well as the main barrier to water leaking towards the interior. The flashing is typically positioned in between the brick veneer and the shelf angle and outside the exterior sheathing. It must extend beyond the face of the brickwork.

Figure 2-4 --- Exterior Elevation of BV/SS Wall Showing Exterior Sheathing and Ties

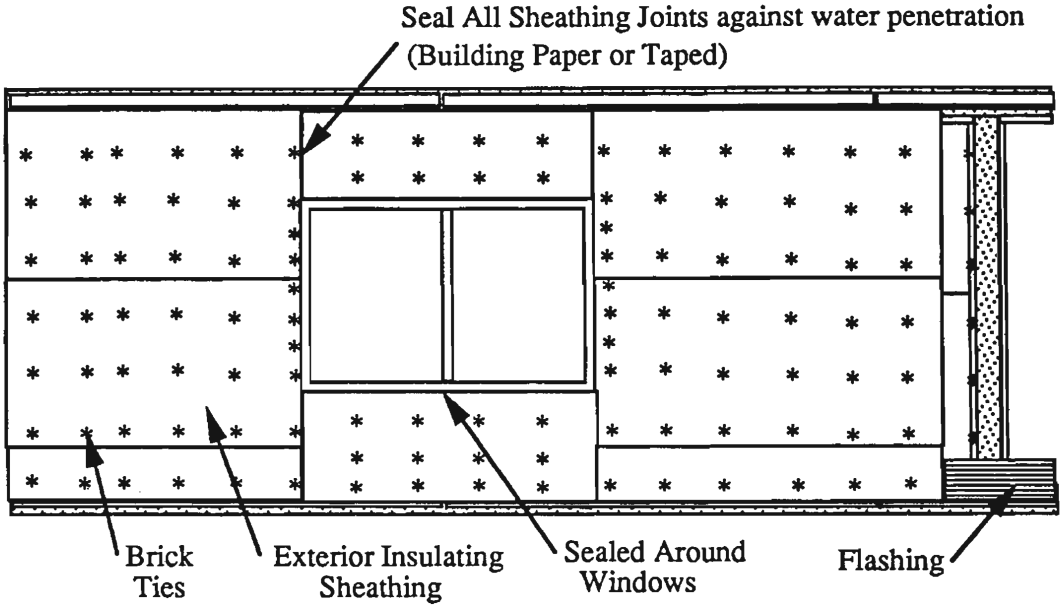
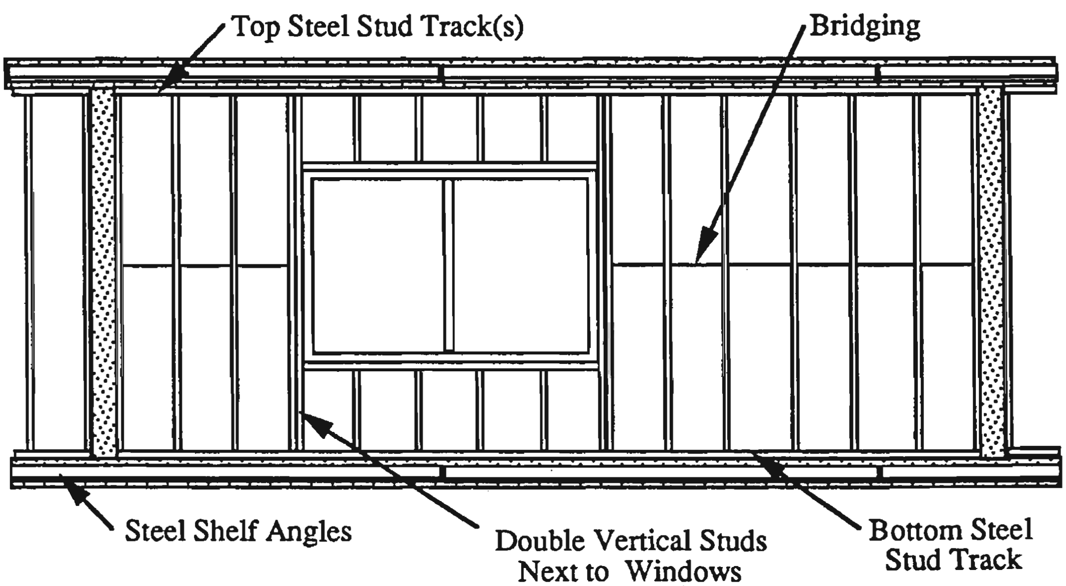


Figure 2-5 --- Exterior Elevation of BV/SS Wall System Showing Steel Studs



The likely location of brick ties are shown on Figure 2-4. Depending on the strength characteristics of the tie, the ties will be installed at either 400mm (16") or 600mm (24") spacing horizontally, and either 400mm (16") or 600mm (24") spacing vertically. Additional ties can be placed within 200mm of windows and other openings to accommodate the higher tie loads. Tie density should also be increased near the top of the wall as the distribution of lateral force is not uniform. It is essential that the ties be properly designed and the appropriate tie be chosen and properly connected. Tie problems are common. For example, in low-rise buildings it has been customary to use corrugated metal ties that transferred compression load first to the exterior sheathing and then to the studs. However, if the sheathing is relatively soft (as is the case with a number of insulating sheathings) the tie connection is flexible and much weaker than expected.

Ties should be securely fastened directly to the stud either to the flange or to the web. The ties that connect to the web of the steel stud, called wrap-around or bayonet ties, have two advantages in that (i) the connection point is much closer to the shear centre of the studs and any tendency to twist or for the flange to bend is reduced and (ii) the sheet-metal screws are in shear rather than tension. The choice of the wrong tie, poor installation, poor construction, corrosion, gross incompetence are some of the possible causes of problems with the lateral support system. Tie problems are common and it is for this reason that this study is directed at developing a retrofit or supplemental procedure for the existing tie system.

Figure 2-5 is an exterior elevation with the exterior sheathing, ties and flashing removed. The steel stud backup is typically enclosed by the floor slabs and the side walls as shown in Figure 2-4, except that sometimes the steel stud backup will be continuous across walls. The steel studs are typically spaced at 400mm (16") o.c. although spacings of 600mm (24") have also been used. At present a steel stud thickness of 1.22mm (0.048" or 18 gauge) is recommended. However the 0.91mm (0.036" or 20 gauge) thickness is still used especially in apartment buildings. Studs as thin as 26 gauge have been used in some buildings[9]. The vertical steel studs fit into the top and bottom tracks which are also made of cold formed steel. The tracks are fastened to the floor slabs preferably using expansion anchors not shot-in anchors. The vertical steel studs are fastened to the bottom track with screws on both the interior and the exterior or by welding if the steel is at least 1.22mm thick. The top track connection is a special one as it must permit vertical movement. There are various methods of making the steel stud system free to translate vertically, the most common methods are what has been termed the free floating stud and nested tracks. Bridging, as shown in Figure 2-5, reduces twisting of the cross section under load and allows the flexural capacity of the steel stud sections to be attained.

If the brick veneer is supported by shelf angles, the brick veneer will be longer than the backup system by about the thickness of the floor slab which is typically 200mm to 250mm (8" to 10") thick. In buildings with deep spandrel beams the brick veneer may be 500 to 800mm longer than the vertical steel studs. The shelf angles are fastened to the floor slabs with welded or bolted inserts. The proper design and installation of the shelf angles is important as any rotation or deflection of a shelf angle can induce excessive stresses in the brick veneer.

While it may have been customary to refer to the BV/SS wall system as a non-load bearing infill and to consider it to be non-engineered component of the building, the extent and the cost of BV/SS problems has forced reconsideration. As should be evident from this description the BV/SS is, in fact, a relatively complex physical sub-system that requires the attention of competent building professionals.

2.2 General Requirements for BV/SS Wall Systems

All types of exterior walls in buildings must satisfy many different requirements. These may include making the building profitable, safe, beautiful, warm or cool, and inexpensive to maintain. The primary requirement will differ depending on the viewpoint of the owner, the tenant or the architect. One common and fundamental requirement is that the wall be structurally sound. This structural requirement is often thought of as a strength criterion. However, structural safety is not strictly limited to strength and the structural performance requirements for these so-called non-load bearing walls are numerous and may not even seem to be structural. Figure 2-6 is an attempt at outlining the structural performance requirements for a wall.

Design codes and philosophies separate the structural considerations into the categories of safety and serviceability. Loadings pertaining to structural serviceability include the usual physical loads such as wind but also differential movement, moisture, and temperature. Failure to meet structural serviceability requirements is not necessarily the same as structural failure. The consequences may be flexurally cracked masonry, water leakage in the wall or corrosion, none of which constitute an immediate safety problem.

A wall that cracks has not failed structurally until there is a risk of bricks falling. Typically much of the cracking that occurs in masonry walls is not of concern with regards to the immediate structural safety of the wall. Over time increased water penetration may give rise to concern about structural safety.

Typically serviceability requirements have been concerned with areas that do not threaten the public safety, at least not in the short term. Cracking of masonry and interior finishes are deemed to be aesthetically unpleasant. However, in the design of BV/SS walls the serviceability requirements also ensure the long-term safety of the wall. Water penetration through the brick veneer may increase if cracking of the masonry occurs. BV/SS wall systems are particularly susceptible to corrosion damage due to increased moisture within the wall system as the backup system is made of steel. Obviously the tie systems are also vulnerable to corrosion damage. This increased vulnerability to moisture of BV/SS changes cracking from being a serviceability concern to being a long term safety concern.

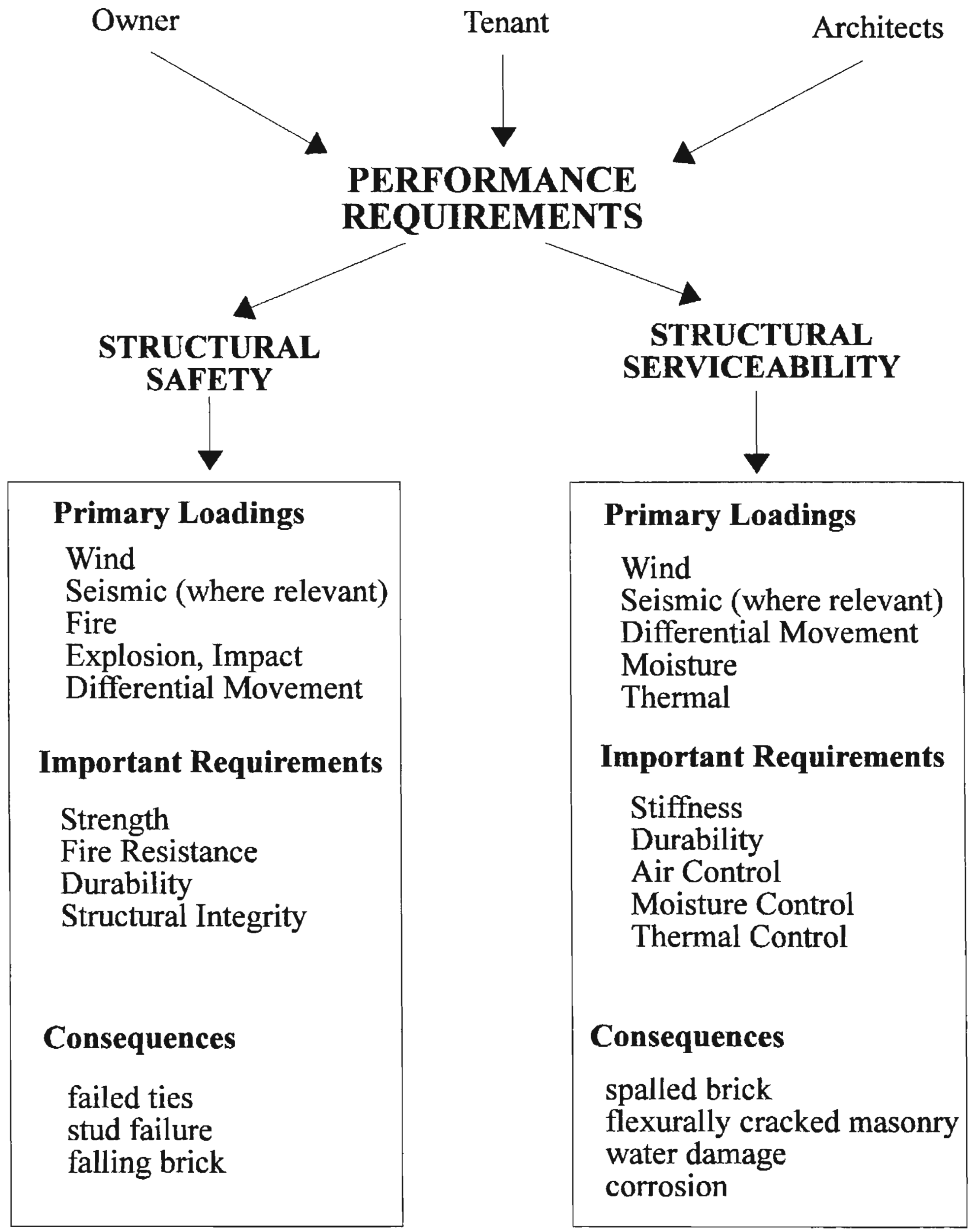


Figure 2-6 --- Structural Performance Considerations for an Exterior Wall System

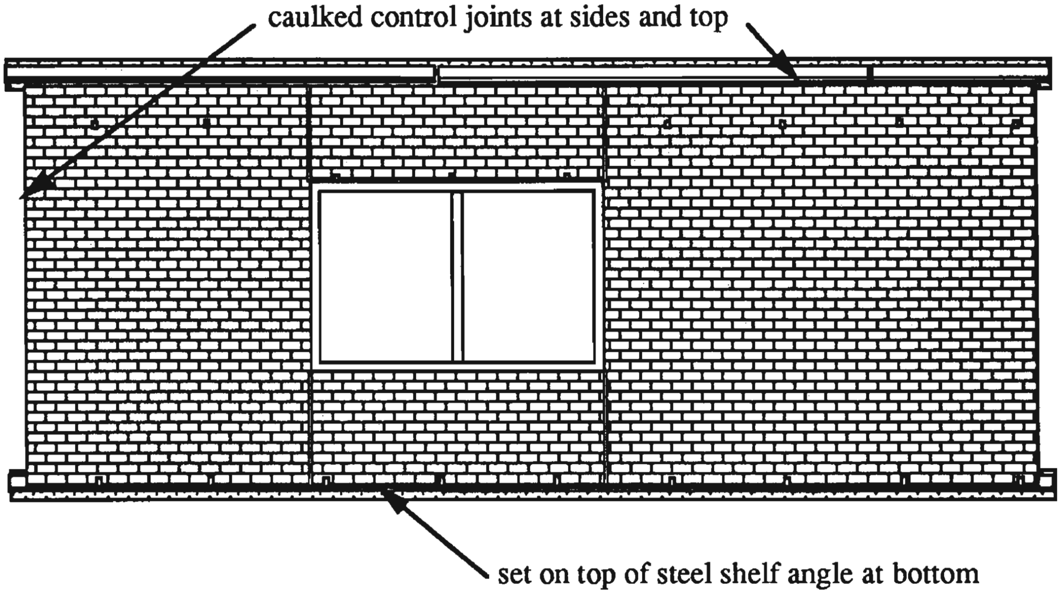
2.3 Structural Response of BV/SS Wall Systems

Brick veneer/steel stud infill is a relatively complicated means of providing an exterior wall. The properties of each component, the connection of these components and the boundary conditions are all important factors that influence structural behaviour. It is necessary to consider the performance of the entire wall system as well as that of each component.

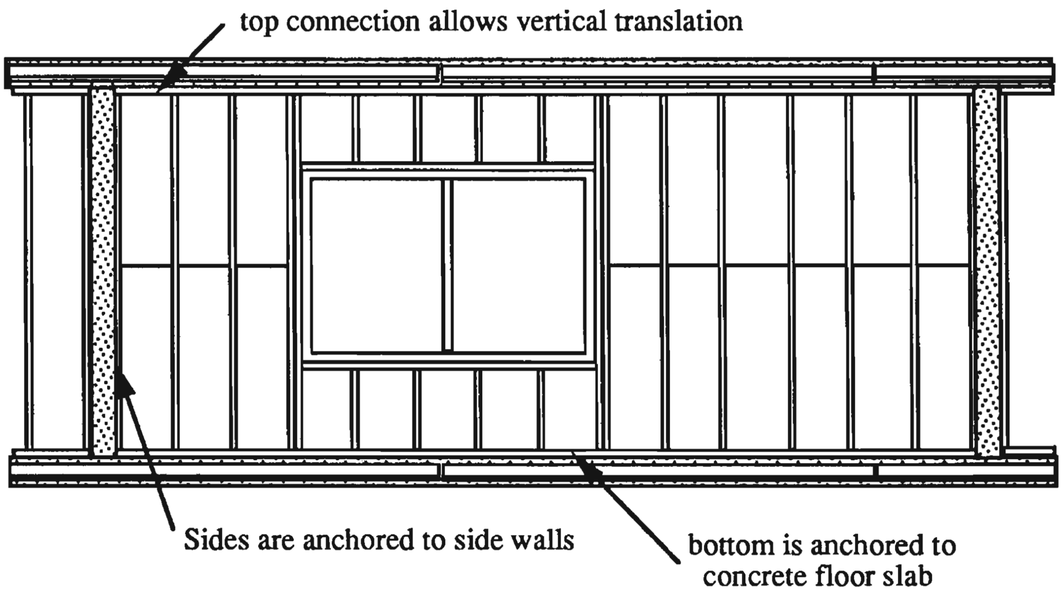
Because of the nature of the boundary conditions, the presence of openings and the nature of tie response, the design of a BV/SS wall system should, in theory, be based on three-dimensional analysis. Figure 2-7 illustrates the boundary conditions for the brick veneer and the steel stud wythes of a wall. The brick veneer is simply supported on top and unsupported at the control joints at the sides and top. The steel studs may not be spaced evenly due to openings such as windows. The boundary conditions for the steel stud framing are : simply supported on the bottom and sides (at the cross-walls) and hinged but free to translate vertically at the top. Modeling of the steel stud framing is complicated by the flexibility of the stud to track connections as there can be some horizontal displacement. For accurate analysis three-dimensional analysis is necessary. However, little three-dimensional analysis has been done, it is not only expensive but the properties of this wall system are very difficult to model. Designers usually resort to analyzing a width of representative wall.

An idealized model of the cross-section of the wall is given in Figure 2-8. This idealized model contains only the components that determine the structural performance of the BV/SS wall, such as the brick veneer, the steel stud and the support and tie system. There have been numerous versions of this 2-dimensional model but the one shown is state-of-the-art. This model accounts for the flexibility of the stud to track connections by using translational springs. Also the top brick veneer joint is modeled as a translational spring.

Figure 2-7 --- Boundary Conditions of BV/SS Wall System

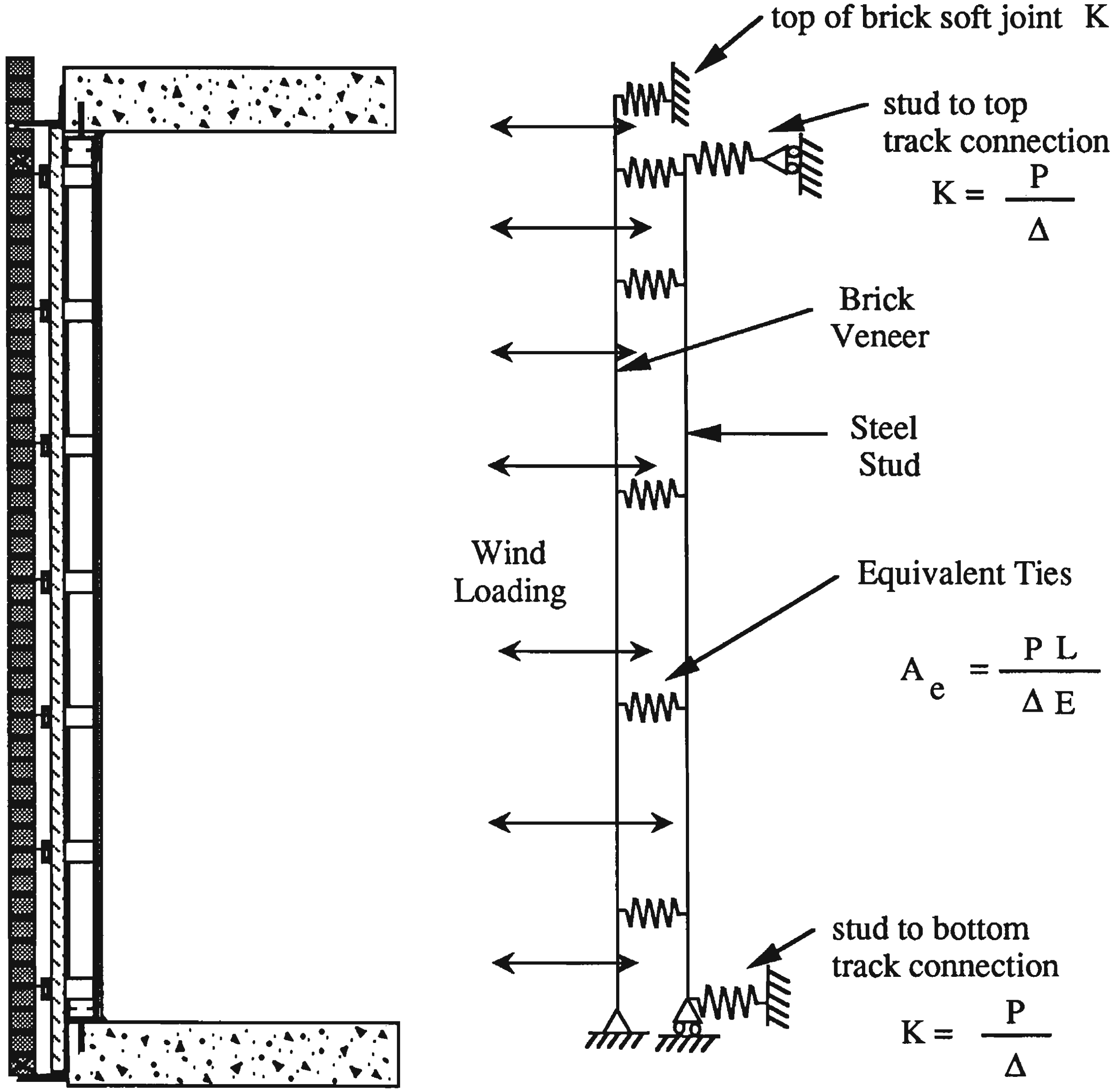


(i) The Brickwork (BV)



(ii) The Steel Stud Backup (SS)

FIGURE 2-8 --- TYPICAL TWO-DIMENSIONAL STRUCTURAL MODEL



CROSS-SECTION OF BV/SS WALL

Model
Effective Tie Areas
Flexible BV top
Flexible Stud Track

The tie system is represented by equivalent members which will deflect under loads. An equivalent member is used to include the effects of tie deformation as well as the stiffness of the tie to stud connection.

The loads on a BV/SS wall may be direct (or indirect) wind, seismic, or differential movement. Some lateral loads may be either push (pressure) or pull (suction). The magnitude and frequency of the wind loading varies with time, over the height of the building and with location, e.g., corners or reveals. BV/SS walls are designed to be rainscreen walls with weep and vent holes and an air space behind the brick. It is claimed that across the screen instantaneous equalization of lateral wind pressure will occur. In fact some degree of pressure equalization or, preferably, pressure moderation will occur. In theory equalization of the pressure difference across the brickwork means that there is no wind load on the brick veneer and, therefore, the total pressure difference across the entire wall must be taken by the backup wythe. In which case the wind load would be acting on the air barrier in the backup wythe. For design purposes the wind load on the brick veneer must be considered to vary between 0 and 100 % of the code designated value.

Note however that the brick veneer and the steel stud must behave with some degree of stucturally composite action as they are connected by the tie system. Moreover at high loads the brick veneer may crack, typically in the middle third of the wall. This type of cracking is not the small micro cracking that occurs in masonry walls due to shrinkage but flexurally induced cracking that affects the load carrying capability of the wall. The flexural stiffness of the veneer is reduced significantly. Before cracking the brick veneer may carry 60 to 90 % of the exterior stagnation pressure depending on the geometry and relative stiffness of the two wythes and their interconnection. After flexural cracking the steel stud wythe must take the majority of the load on the wall system.

Currently it is recommended that not only the brick veneer but also the steel stud framing be designed to accommodate 100 % of the design wind load. Current structural codes suggest that after cracking the contribution of the brick veneer is negligible and the backup steel studs will carry the full design load. This is good design practice for the present as it provides redundancy. Until the behaviour of the brick veneer is better understood and modeling techniques are improved and verified, this design methodology is warranted.

Typically safety requirements for steel stud framing are readily satisfied and it is serviceability that control the design. The design philosophy related to serviceability focuses on the control of the initiation or the width of cracks in the brick veneer once cracking has occurred. This control is deemed important as the potential for increased water penetration will jeopardize the performance and, later, the safety of the wall system. Designers have attempted to minimize the deflection of the brick veneer and thus minimize the crack widths.

The proposed changes to CAN3-S304[10] outline these special provisions for walls with flexible backup systems. A backup wall is defined as flexible if the backup wall stiffness (EI) is less than 2.5 times the uncracked stiffness (EI) of the brick veneer. Figure 2-9 shows the relative stiffness for various gauges for the steel stud framing and stud spacings relative to brick veneer with various moduli of elasticity. It is clear from this figure that, in those instances when the steel stud is the backup, the backup wythe must be considered to be flexible. In comparison Figure 2-10 shows that while 190mm and 240mm hollow concrete block backup wythes are not flexible, some 140mm and 90 mm hollow concrete block backup wythe are flexible as the stiffness ratio is less than 2.5. With concrete block grouting the cores and adding reinforcement may be an effective way of making the backup non-flexible.

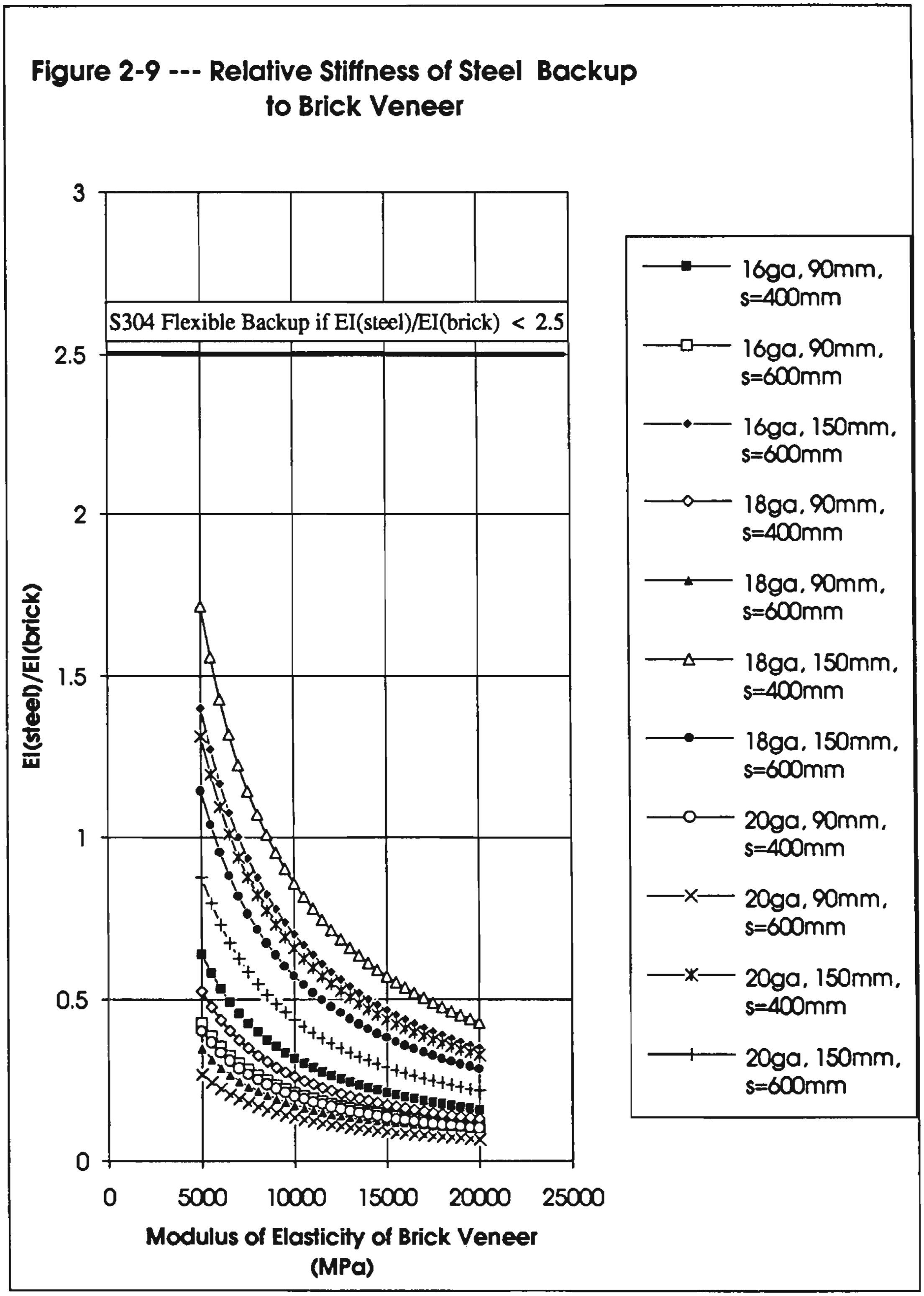
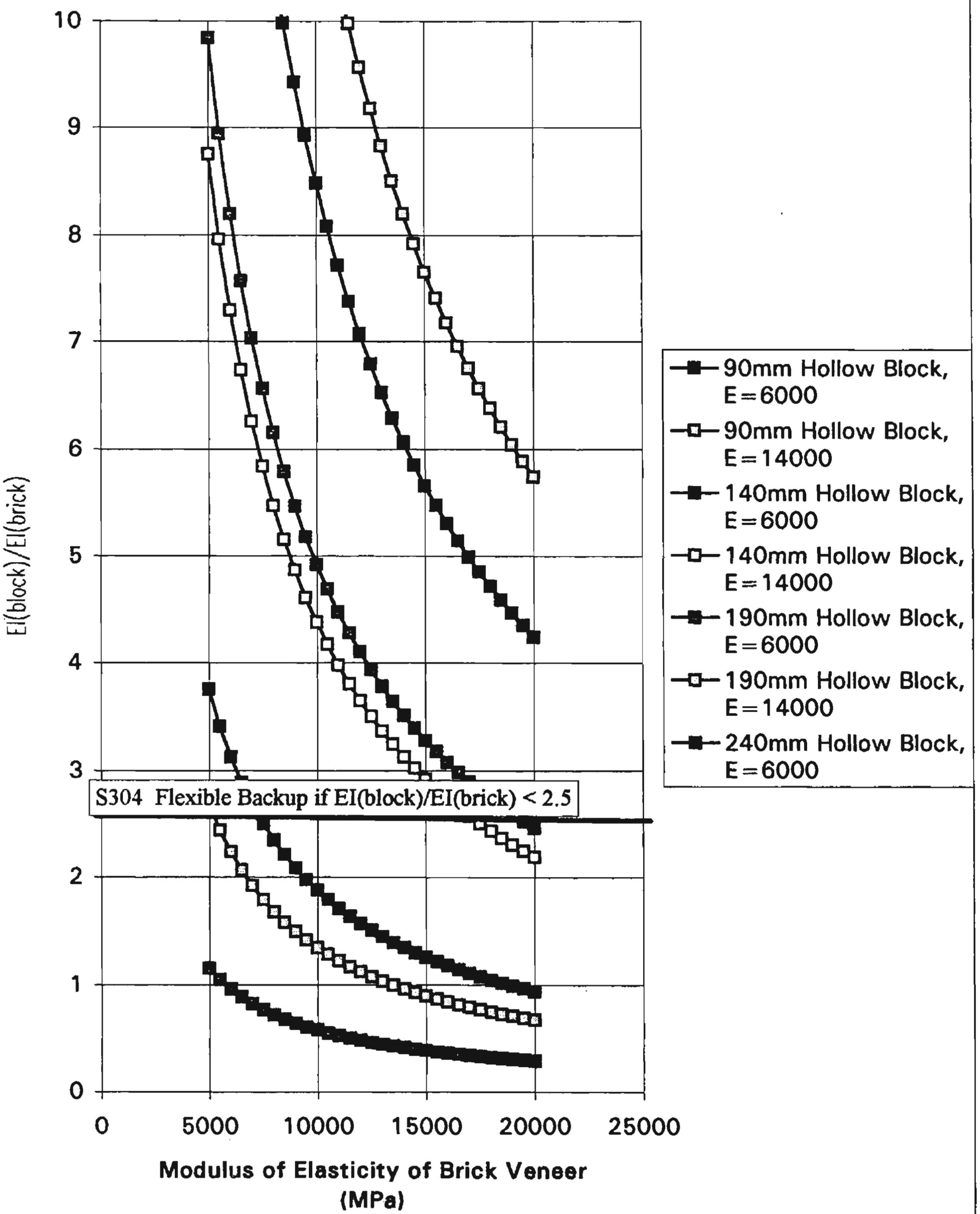


Figure 2-9 --- Relative Stiffness of Steel Backup to Brick Veneer
S304 Flexible Backup if EI(steel)/EI(brick) < 2.5
EI(steel)/EI(brick)
0
0.5
1
1.5
2
2.5
3
0
5000
10000
15000
20000
25000
Modulus of Elasticity of Brick Veneer (MPa)
16ga, 90mm, s=400mm
16ga, 90mm, s=600mm
16ga, 150mm, s=600mm
18ga, 90mm, s=400mm
18ga, 90mm, s=600mm
18ga, 150mm, s=400mm
18ga, 150mm, s=600mm
20ga, 90mm, s=400mm
20ga, 90mm, s=600mm
20ga, 150mm, s=400mm
20ga, 150mm, s=600mm

Figure 2-10 --- Relative Stiffness of Block Backup to Brick Veneer

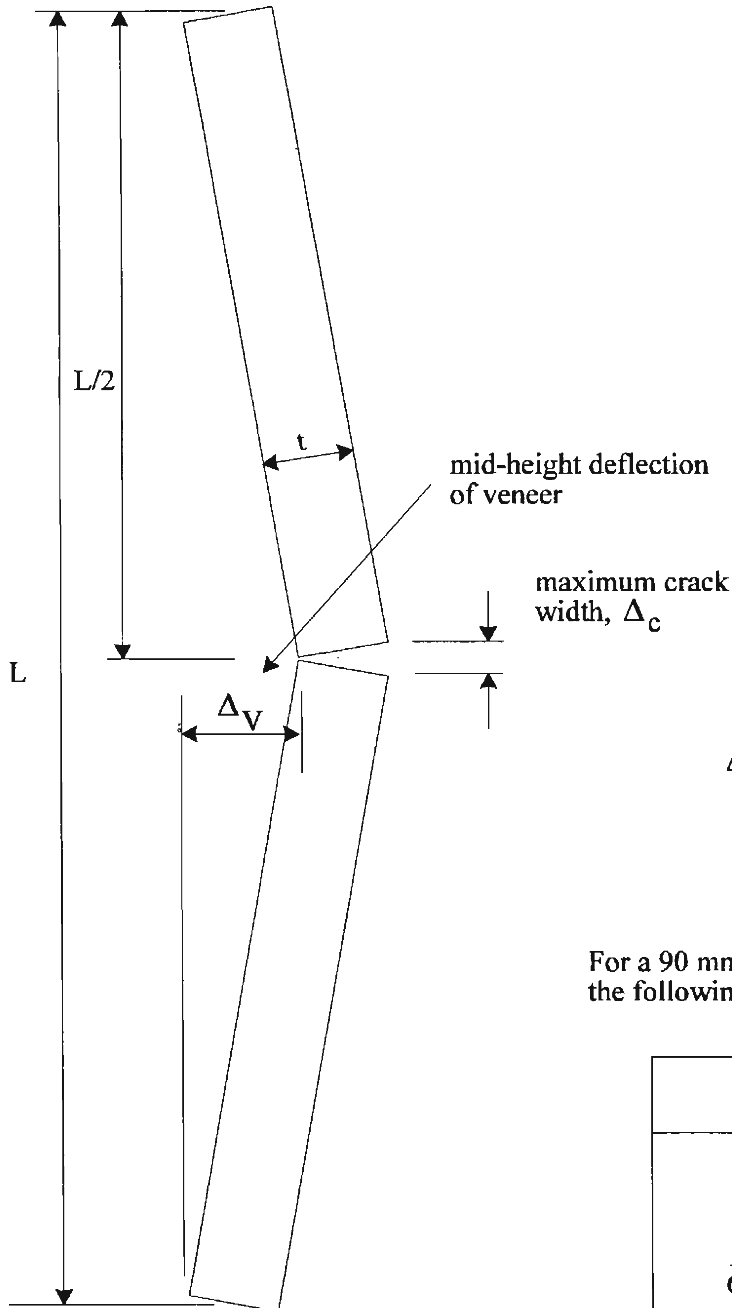


McGrath and Drysdale[11] stated that for the design of BV/SS walls with flexible backup :

> *"For such cases the veneer deflection under specified loads is limited to L/600. This criterion is assumed to be met if the stud deflection does not exceed L/720 and the tie deflection due to one half the maximum mechanical play and a tension or compression load of 0.45 kN does not exceed 1.0 mm."*

The L/600 limit was arrived at by considering the simple structural model to predict crack widths described by Drysdale in the CMHC advisory document[12] and shown in Figure 2-11. The advisory document recommends a value of L/720 to limit the maximum crack width to 0.5mm or the average crack width to 0.25mm, while the McGrath and Drysdale paper recommends L/600.

Meeting two criteria, L/720 for the steel stud backup alone under full design load and limiting the tie deflection, implicitly limits the brick veneer deflection to L/600. Presumably crack widths will be small enough so that only a limited increase in water penetration will result.



$$\Delta_c = 4\,\Delta_V \frac{t}{L}$$

$$\text{if } \Delta_V \text{ is } \frac{L}{C}$$

$$\Delta_c = 4\,\frac{t}{C}$$

For a 90 mm thick brick and typical C values the following maximum crack widths result :

$\frac{L}{C}$	Δ_c
$\frac{L}{360}$	1 mm
$\frac{L}{600}$	0.6 mm
$\frac{L}{720}$	0.5 mm

Figure 2-11 --- Schematic Drawing for Simplistic Calculation of Crack Width in Brick Veneer (adapted from CMHC advisory document)

2.4 Structural Requirements for the Brick Veneer

The BV/SS is a composite structural system with regard to out-of-plane loading. It has been suggested that the share of the lateral load taken by the veneer can be approximated by the ratio of the simple flexural stiffness of the brick veneer to the simple flexural rigidity of the total wall, veneer and studs,[13] i.e., the percentage of load, w_{BV}, carried by the brick veneer is given as :

$$w_{BV} = 100\left[\frac{EI_{BV}}{EI_{BV} + EI_{SS}}\right]$$

This expression assumes that the length of both wythes are the same and that all the boundary conditions for both wythes are the same. For a BV/SS wall neither of these conditions apply. Also the use of a flexural stiffness ratio to proportion lateral load requires that the deflection profile of the two wythes be the same. This is only theoretically true for walls connected by stiff ties with identical wythes. In reality the load proportion is influenced by the degree of pressure equalization or, preferably, pressure moderation that occurs across the screen. It does make a difference how the direct load is applied and subsequently what effect composite structural action has on this initial distribution. It follows that the use of this ratio is not correct and likely to be misleading. Nevertheless the use of the ratio of flexural rigidities is useful as an approximate method of assessing the load distribution.

On the basis of this ratio Figure 2-12 shows the percentage of load, w_{BV}, carried by the brick veneer for various types of steel stud gauge and spacing and for various modulus of elasticity of brick. From Figure 2-12 it is evident that, according to this simplistic method of load distribution, the modulus of elasticity of the brick veneer and the type of stud have a significant effect on the amount of load attributed to the brick veneer. For the popular 90mm, 20 gauge stud at 400 mm spacing the range of w_{BV} is from 70 to 90%. For 90mm, 18 gauge stud at 600 mm the range of w_{BV} is 65 to 88%.

Both Chin et al.[14] and Brooks[15] have concluded that this method of distribution of lateral load will result in errors. The boundary conditions, the tie stiffness and the length of the wythes should all be considered. Brooks concluded that distributing the lateral load on a flexural rigidity basis is a reasonable approximation for stiff ties. With flexible ties an increase in the stiffness of the veneer had little effect on the lateral load distribution. The largest single factor affecting the distribution of lateral load was the degree of restraint at the horizontal control joint between the shelf angle and the top of the brick veneer. Generally as the top restraint decreased, the stress level at the middle of the brick veneer wythe decreased. With no top restraint the stress in the veneer was found to be considerably less for flexible ties than for stiff ties. Chin obtained similar results to Brooks.

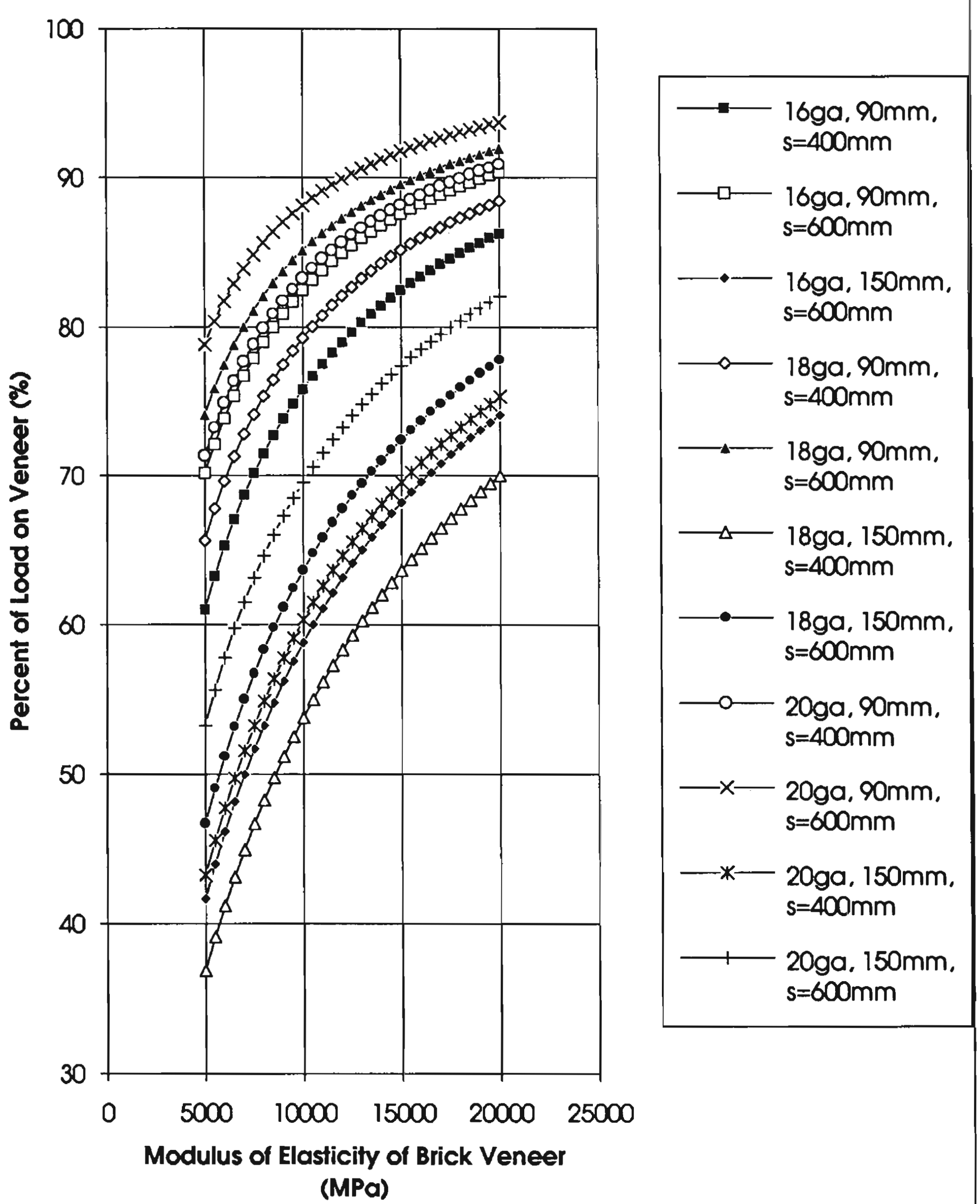
If and when a structural crack occurs in the brick veneer, lateral load redistribution occurs with the backup wythe taking more of the load. Structural cracking of the veneer does not constitute structural failure. Instead, cracking of the veneer may effect the long-term safety of the wall system by potentially increasing water penetration. It needs to be determined whether the brick veneer will actually crack under the anticipated service loadings and, if so, whether increased water penetration will, in fact, result.

2.4.1 The Initiation of Flexural Cracking

Masonry may crack from wind loading, vibration, settlement, or volume changes with changes in temperature, moisture, salt crystallization, and corrosion. In the evaluation of cracking in BV/SS walls all of these causes are important, however most of the research has dealt with cracking due to wind loading. For the purposes of evaluating the research dealing with cracking under wind loading, the categorization of structural and non-structural cracks is useful. The likely width of a crack, the classification of all types of cracks and the implications of cracking for water penetration must also be considered.

A crack occurs in masonry when the tensile stress exceeds the tensile or flexural bond strength. This crack will usually occur at the interface between the mortar and the brick. Unlike the cracking of concrete which usually occurs at numerous locations, masonry will typically only develop one crack when subjected to flexure. Based on full scale experimental testing the location of the horizontal flexural tension crack typically appears

Figure 2-12 --- Percentage of Load Carried by Veneer According to Flexural Rigidities



within the middle third of the height of the brick veneer, wherever the moment is greatest.[16,17]

Masonry codes specify the allowable flexural stress values for clay brick masonry and concrete block masonry for different mortar types and alignment i.e., depending on whether the loading is parallel or perpendicular to the bed joint of the mortar. The allowable flexural stresses given in the Uniform Building Code[18] and the CSA 304.1[19] are shown in Table 2-1. Allowable values were developed from tests on full scale walls using air bags (ASTM E 72). The number of tests were limited due to expense. Because of variability of the test results, a factor of safety of 3 or 4 against actual flexural bond strength was used to establish the allowable tensile stresses. These allowable values are therefore a conservative indicator on the flexural strength of masonry veneers and should not be taken as the stress at which cracking will occur.

Recently the code values have come under much scrutiny, as many believe that the allowable stress values are not reasonable and put masonry at a disadvantage to other building materials. This claim is substantiated by numerous experimental studies that show the actual flexural strength of masonry, although highly variable, to be well above the allowable code stresses. A recent paper by Ghosh is one of the most thorough summaries of the experimental work related to the flexural bond strength of masonry.[20] Table 2-2 summarizes the results from some of these studies. The code recognizes that brickwork is anisotropic as the allowable tensile stresses perpendicular to the bed joint is twice that parallel to the bed joint. Although there are few studies on the anisotropic behaviour of brickwork, they indicate ratios of orthogonal strengths of 3 to 4[21] and even up to 7[22].

From Table 2-2 it is apparent that the actual tensile or flexural bond strengths are many times larger than the allowable stresses in the masonry codes. For portland cement with lime mortars (PCL), the actual flexural bond strength is typically between 0.55 and 1.1 MPa. For masonry cement mortars the actual flexural bond strength is typically between 0.4 and 0.68 MPa.

Table 2-1 Allowable Tensile Stresses for Unreinforced Masonry Walls in Flexure

- **UBC, 1988 Edition**

Type of Mortar	Type of Unit	Tension Normal to Bed Joint Clay Units MPa (psi)	Tension Normal to Head Joint Clay Units MPa (psi)
PCL Type M,S	Solid Unit	0.248 (36)	0.496 (72)
	Hollow Unit	0.152 (22)	0.310 (45)
PCL Type N	Solid Unit	0.186 (27)	0.372 (54)
	Hollow Unit	0.117 (17)	0.234 (34)
MC Type M,S	Solid Unit	0.088 (18)	0.248(36)
	Hollow Unit	0.076 (11)	0.159 (23)
MC Type N	Solid Unit	0.097 (14)	0.186 (27)
	Hollow Unit	0.055 (8)	0.117 (17)

- **CSA S304.1**

Type of Mortar	Type of Unit	Tension Normal to Bed Joint Clay Units MPa (psi)	Tension Normal to Head Joint Clay Units MPa (psi)
Type M or S	Solid Unit	0.25 (36)	0.50 (72)
Type N	Solid Unit	0.19 (28)	0.39 (57)

Table 2-2 -- Summary of Flexural Bond Strengths from Experimental Studies

- **Construction Technology Laboratories, Stokie IL, 1989[23]**

Type of Mortar	Tension Normal to Bed Joint - Clay Brick Units				
	Low Value MPa (psi)	Average Value MPa (psi)	High Value MPa (psi)	UBC Allowable Stress MPa (psi)	Ratio of Average to UBC
PCL Type M	0.524 (76)	1.165 (169)	2.268 (329)	0.152 (22)	7.68
PCL Type S	0.427 (62)	1.041 (151)	1.620 (235)	0.152 (22)	6.86
PCL Type N	0.317 (46)	0.531 (77)	0.696 (101)	0.117 (17)	4.53
PC/MC Type M	0.386 (56)	0.876 (127)	2.006 (291)	0.076 (11)	11.55
PC/MC Type S	0.324 (47)	0.758 (110)	1.131 (164)	0.076 (11)	10.00
MC Type M	0.248 (36)	0.607 (88)	1.120 (174)	0.076 (11)	8.00
MC Type S	0.359 (52)	0.593 (86)	1.000 (145)	0.076 (11)	7.82
MC Type N	0.359 (52)	0.524 (76)	0.855 (124)	0.055 (8)	9.50

* PC -Portland Cement, L - Lime, MC - Masonry Cement

** Bond Wrench Test[24] was used in these tests

- **McMaster University, 1985[25]**
  - Bond Wrench Test, 475 tests normal to bed joint
  - flexural strengths ranging from 0.29 to 0.97 MPa (42 to 141 psi)
  - PCL Mortars had greater flexural strengths than other mortars

- **University of Texas at Arlington, 1989[26]**

Type of Mortar	Tension Normal to Bed Joint - Clay Brick Units
PCL Type M	0.556 to 0.655 MPa (81 to 95 psi)
PCL Type S	0.563 to 0.871 MPa (82 to 126 psi)
MC Type M	0.117 to 0.722 MPa (17 to 105 psi)
MC Type S	0.072 to 0.407 MPa (10 to 59 psi)

* bond wrench test used

** variation due to solid and hollow bricks and testing at 28 days and 6 months

- **Clemson University, 1986[27]**

Type of Mortar	Tension Normal to Bed Joint - Clay Brick Units
PCL Type M	0.793 MPa (115 psi)
PCL Type S	0.910 MPa (132 psi)
PCL Type N	0.979 MPa (142 psi)
MC Type M	0.448 MPa (65 psi)
MC Type S MC Type N	0.545 MPa (79 psi) 0.683 MPa (99 psi)

Many factors affect the flexural bond strength of masonry, however not all researchers agree on which. Factors that are generally agreed upon are :

1. cementitious material, whether PCL, MC or PC/MC
2. mortar type, Type M, S, or N
3. air entrainment in mortar
4. surface texture of brick units
5. atmospheric conditions during curing
6. admixtures such as superplasticizers or latex modified mortars
7. workmanship

Properties that are reported to have some influence on flexural bond strength are :

1. IRA, initial rate of absorption of the brick
2. flow of the mortar
3. fineness of masonry cement and/or sand

The most controversial issue is the IRA of the bricks. Some researchers[28,29,30], have found a direct correlation between IRA and flexural bond strength while others[31,32,33] have found little correlation.

Full scale walls have also been tested. The cracking loads for some of these studies are shown in Table 2-3. These studies incorporated many different types of brick, stud and tie. Despite the variability in wall make-up, the cracking loads are all above typical design loads. This indicates that even for brick veneer walls with flexible backup, cracking under design service loads is unlikely. There is also a significant margin of safety against collapse as the recorded values were 4 to 7 times the typical design load. As these walls were new walls it is not known whether walls that have been exposed to long term weathering would give comparable results.

The dilemma is as follows. In order to be statistically consistent and conservative, a relatively low value of flexural tensile strength has to be used. However, this value is so low that it militates against the use of BV/SS walls. What then is the appropriate serviceability criterion to ensure good performance and facilitate the use of BV/SS wall systems? We do not have an answer to this design problem.

Table 2-3 - Summary of Full Scale Wall Testing

	Clemson[34]	Clemson[34]	University of Alberta[35]	University of Alberta[35]	University of Alberta[36]
	Wall 1	Wall 2	Wall 1	Wall 2	Wall 3
BV height	2845 mm	2845 mm	3200 mm	3200 mm	3200 mm
BV length	1220 mm	1220 mm	1220 mm	1220 mm	1220 mm
BV thickness	89 mm	89 mm	76 mm	76 mm	76 mm
BV IRA	7 to 13	7 to 13	-	-	-
bond strength	0.614 MPa	0.952 MPa	-	-	-
Mortar	PCL, TypeS	PCL, TypeS	-	-	-
Joint at Shelf Angle	comp. filler	comp. filler	Flexible	Flexible	Flexible
SS height	2400 mm	2400 mm	3000 mm	3000 mm	3000 mm
SS gauge	20	20	18	20	*
SS width	89 mm	89 mm	90 mm	150 mm	*
SS spacing	600 mm	600 mm	400 mm	400 mm	*
Cavity	25 mm	25 mm	50 mm	50 mm	50 mm
Sheathing	Gypsum	Gypsum	Gypsum	Gypsum	Gypsum
Bridging	1 row, 16 ga., Web	1 row, 16 ga., Web	-	-	-
Tie Type	DW10 adjustable	DW10 adjustable	Bloc-Lok 319 adjust.	Bloc-Lok 319 adjust.	22 ga. corrugated strip ties
Stiffness, comp Stiffness, tens	1.3 kN/mm 0.26 kN/mm	1.3 kN/mm 0.26 kN/mm	0.92 kN/mm	0.85 kN/mm	0.62 kN/mm
Tie spacing, horiz.	600 mm	600 mm	400 mm	400 mm	400 mm
Tie spacing, vert.	400 mm	400 mm	533 mm	533 mm	533 mm
Loading Method	Air	Air	Air Bag	Air Bag	Air Bag
Load Type	Pressure	Suction	Pressure	Pressure	Pressure
First Crack Load	2.5 kPa 2.5 kPa 1.25 kPa	2.7 kPa >3.5 kPa >3.5 kPa	> 2.0 kPa	> 2.0 kPa	1.75 kPa
Second Crack Load	-	-	-	-	-
Failure Load	-	-	-	-	3.46 kPa

Table 2-3 cont'd - Summary of Full Scale Wall Testing

	Univ.of Alberta[36]	McMaster University[37]	McMaster University[37]	McMaster University[37]	McMaster University[37]
	Wall 4	Wall 1	Wall 2	Wall 3	Wall 4
BV height	3200 mm	2747 mm	2747 mm	2680 mm	2680 mm
BV length	1220 mm	5200 mm	5200 mm	5200 mm	5200 mm
BV thickness	76 mm	90 mm	90 mm	90 mm	90 mm
BV IRA	-	21.5	21.5	21.5	21.5
bond strength	-	0.72 MPa	0.89 MPa	0.78 MPa	0.68 MPa
Mortar	-	PC/MC, Type S	PCL, Type S	PCL, Type S	PC/MC, Type S
Joint at Shelf Angle	Flexible	Flexible	Flexible	Flexible	Flexible
SS height	3000 mm	2590 mm	2590 mm	2590 mm	2590 mm
SS gauge	-	18	18	2 @18	18
SS width	-	90 mm	90 mm	90 mm	90 mm
SS spacing	-	406 mm	406 mm	813 mm	406 mm
Cavity	50 mm	25 mm	25 mm	50 mm	50 mm
Sheathing	Gypsum	Gypsum	Gypsum	Gypsum	Gypsum
Bridging	-	1 row, 18 ga., Web	1 row, 18 ga., Ext.	1 row, 18 ga., Web	1 row, 18 ga., Web
Tie Type	T-Ties	Wire Loop Adjustable	Self Drill Tie & pintle	Bayonet and V-Tie	Double Leg Adjustable
Stiffness, tens	0.78 kN/mm	0.43 to 0.60 kN/mm	0.52 to 0.53 kN/mm	0.69 to 0.78 kN/mm	0.36 to 0.61 kN/mm
Tie spacing, horiz.	400 mm	406 mm	406 mm	813 mm	406 mm
Tie spacing, vert.	533 mm	603 mm	603 mm	400 mm	603 mm
Other		side studs free, one way	side studs anchored, two way	inverted backup	window in wall
Load Method	Air Bag	Air Bag	Air Bag	Air Bag	Air Bag
Loading Type	Pressure	Pressure	Pressure	Pressure	Pressure
1'st Crack Load	2.07 kPa	1.4 kPa*	1.6 kPa*	1.2 kPa*	0.8 kPa*
2'nd Crack Load	-	3.8 kPa	4.0 kPa	4.6 kPa	4.0 kPa
Failure Load	4.83 kPa	7.2 kPa	7.2 kPa	4.6 kPa	> 4.0 kPa

* In these tests the cavity was pressurized so the loading was on the backup system

2.4.2 Crack Width

Classifications of cracks, according to their width, have been made by Bidwell[38], Rainer[39] and Kaminetzky.[40] These are shown in Table 2-4.

Table 2-4 Classification of Crack Width in Masonry

Bidwell		Rainer		Kaminetzky		Concrete Practice	
				Negligible	up to 0.1mm	Visible	0.1mm
		Very Slight	up to 1mm	Very Light	0.4mm	Interior Exposure	0.33mm
Fine	up to 1.5mm	Slight	1 to 5mm	Light	0.8 to 3.2mm	Exterior Exposure	0.4mm
Medium	1.5 to 10mm	Moderate	5 to 15mm	Moderate	3.2 to 12.7mm		
Wide	> 10mm	Severe	> 15mm	Extensive	12.7 to 25.4mm		
				Very Extensive	> 25.4mm		

It is interesting to compare these masonry classifications with common practice for reinforced concrete. They involve different orders of magnitude yet, given the extent of moisture problems in masonry, a crack width 2 to 3 times the largest reinforced concrete limit is uniformly regarded as fine, slight, or light. The concrete crack limitation is applicable to beams and slabs in which the dead load part of the service loads remain and crack widths that are constantly open are the main concern.

Grimm states that it is generally accepted that cracks smaller in width to 0.1 mm are negligible in terms of water penetration.[41] Another study recorded maximum crack widths on an actual building over the course of one winter to be 0.075 mm.[42] This study was limited to a very short time span and any displacements were likely to have been due to thermal movements rather than wind loads.

The simple model to predict crack widths and the associated crack width limits shown in Figure 2-11 is claimed to have been validated by Drysdale and Wilson[43]. In the CMHC advisory document this model is correctly termed "simplistic and slightly conservative".

This document goes on to adopt a crack width of 0.25 mm as being acceptable, referring to the fact that this value is consistent with structural concrete practice. Using the 0.25 mm value as an average crack width in the simple rigid body model, the deflection limit of L/720 for the brick veneer was arrived at. From the McMaster study the lateral displacements of the veneer for Walls 1, 2, and 3 at their first cracking pressures of 1.4 kPa, 1.6 kPa and 1.2 kPa were 1.1mm, 1.5mm, and 0.6 mm respectively. This converts to span to deflection ratios of L/2500, L/1800, and L/4470. It would therefore seem that the L/720 limit is not meant to avoid cracking but to limit the width of subsequent cracks. The problem with the use of the L/720 limit is :

1) To initiate a crack requires a wind load of the order of 1.2 to 1.6 kPa but this is generally greater than the 1 in 10 year cladding design pressure.
2) If cracking does occur, it will do so for a wind loading that probably only recurs at 10 year or greater intervals; afterwards the crack will tend to close up.
3) As rain penetration is the main issue, one is really concerned with the dual probabilities of extreme gusting and rain. This does affect the probability of occurrence.
4) Issues such as orientation of the cladding, extent of pressure moderation across the screen, location of the facade, etc., will all tend to diminish the overall significance of this crack and its contribution to rain penetration.

2.4.3 Water Penetration

Some experimental studies have been conducted to determine if cracking of the veneer results in increased water penetration. The number of studies is limited but they seem to indicate that increased water penetration will occur. However, there is conflicting evidence as to how significant is the increase in water penetration because of veneer cracking due to lateral loads. For example a CMHC sponsored study at McMaster University concluded that :

> *"For large pressure differences across the veneer, the rain penetration increased several fold whereas for equal pressures on both sides of the veneer the increased leakage was much less and only fractionally more than for the uncracked section."*

This may be construed to mean different things. For instance it could mean that a pressure equalized rainscreen does work. There seems to be widespread belief that rain will not penetrate through the veneer in BV/SS wall systems if they are designed and built to be fully pressure equalized. However, it has recently been shown in work at the University of Waterloo[44] that in actual buildings pressure equalization is, in a relative sense, rarely attainable. In certain locations, for wind in some directions parts of a wall may be pressure equalized for part of the time; if it is also raining then rain penetration at this windward location then rain penetration will be reduced.

In the Clemson study it was concluded that water permeance did not correlate closely with the magnitude of the applied load. Other research indicates cracks gradually plug up by autogenous healing.[45] Research has also indicated that the majority of water penetration occurs at the head joint not the bed joints.

What is often overlooked is that there exists a significant body of literature that suggests that all brick veneer leaks extensively whether structurally cracked or not. Therefore water will enter the cavity through the brickwork. Brick veneer can leak profusely. However, if the wall is properly designed the water in the cavity will pose as much of a threat to the long term durability of a BV/SS wall as to brick veneer screens with concrete block backup. Instead of focusing so much attention on eliminating or limiting structural cracking of the brick veneer, it is perhaps more important to stress the following priority needs :

1) the proper flashing, clear cavities and proper weep holes, i.e., adequate drainage
2) the proper control of air, moisture and heat flow
3) a moisture seal to the cavity space at the plane of the exterior sheathing so water does not penetrate past this plane.
4) adequate corrosion protection for all metal components in the cavity space

2.5 Structural Requirements for the Steel Stud

It is common design practice for the steel stud wythe to be designed to resist the entire lateral load and to also meet a simple deflection criterion. The steel stud is thus a true "backup" in that the full flexural capacity of the steel stud will only be needed if the brick veneer cannot contribute to resisting the lateral loads.

The reason for having the studs meet a specific deflection criterion under the entire lateral load is to limit the brick veneer deflection and thus avoid or limit cracking[46]. As previously discussed this deflection criterion is of questionable merit as the method used for estimating crack widths is overly simplistic and the reduction in rain penetration, if any, due to structural cracking is not known.

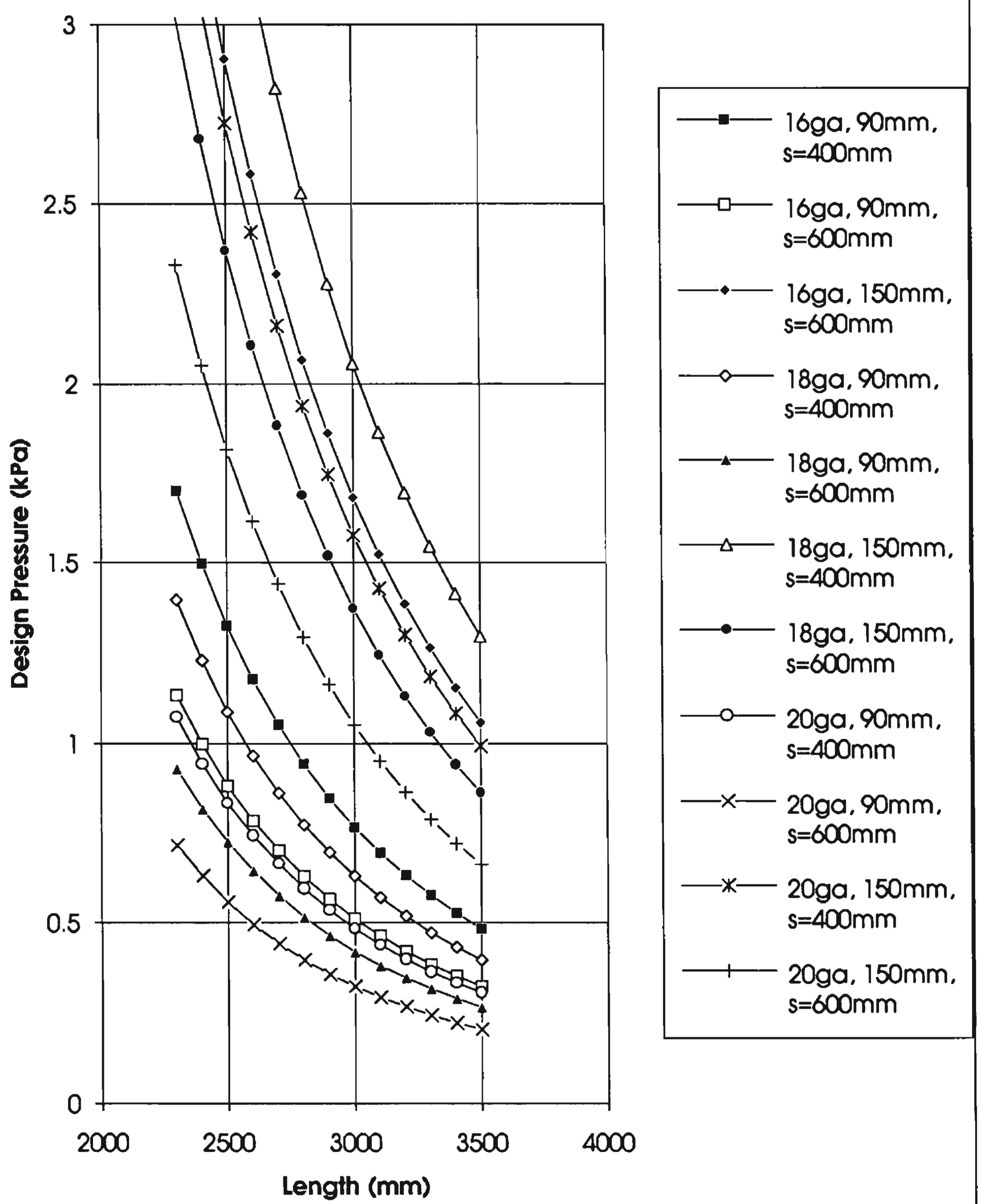
The recommended span deflection ratio has changed numerous times over the years. While steel stud manufacturers give design tables with L/240, L/360, L/600 and L/720, the recommended value in the CMHC advisory document is L/720. The limit suggested by the Brick Institute of America[47] is L/600.

To illustrate one of the problems involved in establishing an optimum span deflection value Figures 2-13 was developed. This figure shows the design pressure for various lengths, gauges and spacing of studs for the L/720 limit. For a typical steel stud wall of length 2500mm and a design pressure of 1 kPa, Figure 2-13 shows that for a L/720 deflection limit 16 gauge, 90 mm studs at 600mm, 18 gauge, 90 mm studs at 400mm and 20 gauge, 90 mm studs at either 400 or 600 mm would not be acceptable. The deflection criterion is quite restrictive.

Although 20 gauge studs have been and still are commonly used, 18 gauge stud is presently being recommended as the minimum thickness.[48,49] The CMHC advisory document cites the following reasons :

1) increased cost of 18 gauge relative to 20 gauge is small
2) current bridging practices are more effective in 18 gauge
3) better screwed and welded connections
4) less susceptible to corrosion
5) less susceptible to damage before installation
6) brick tie to stud connections will be less flexible

Figure 2-13 --- Design Pressure vs. Stud Height for Various Stud Types and Spacing, L/720 Deflection Limit



2.6 Structural Requirements for the Tie System

The behaviour of the tie system is very important to the overall performance of a masonry wall. How important is only now becoming generally known to designers. The choice of masonry ties has typically been considered a detail in a non-engineered wall system designed by architects. Building codes and standards have typically specified minimum strength conditions and set maximum spacing limits in an effort to control tie loads and, presumably, deformations. Masonry ties are now being given much closer consideration as problems such as corrosion, water penetration and cracking of brickwork, have appeared . The much greater use of BV/SS in high-rise construction has made the system more vulnerable to problems, especially moisture problems.

In Canada, before 1984, there was no specific standard for masonry ties. The most complete guidelines for masonry ties were contained in literature published in the Brick Institute of America's Technical Notes on Brick Construction. The Canadian Code for Connectors, CAN/CSA A370-84, was introduced in 1984 and contained Working Stress Design approaches for the design of connectors. Much of the content of the standard dealt with empirical rules for the use of common types of connectors. Some test methods were outlined that were to be used in qualifying new connectors for use. The performance of the tie was based solely on the ultimate strength of the connector.

The CSA code for masonry connectors, CAN/CSA A370, is presently being updated to incorporate limit state design methods similar to those in the design of wood, steel and concrete structures. This new design code will outline specific performance requirements for masonry connectors in terms of safety at factored loads and in terms of serviceability at service loads. The next section of this report will identify and assess the recommended requirements of the CSA code as well as various other sources. These requirements are largely intended for new construction but their relevance to retrofit/remedial situations will also be discussed.

2.6.1 Structural Safety - CAN/CSA A370-93 Requirements

The proposed CAN/CSA A370 standard separates tie connectors into conventional and non-conventional connectors. Conventional connectors have been proven to satisfy minimum strength requirements when used in strict accordance with the provisions of Clause 9 of CAN/CSA A370. Conventional connectors include corrugated strip ties, Z-wire ties, rectangular wire ties, continuous welded ladder and truss ties/reinforcing, Dovetail ties and corrugated Dovetail ties. These ties are accepted as meeting the requirements for safety if the provisions specifying maximum tie spacing, installation procedures and maximum cavity widths are followed. For all other connectors the strength requirements have to be met by laboratory or in-situ testing.

A new clause in the proposed CAN/CSA A370-93, Clause 7.1, stipulates a minimum ultimate strength for tie connectors of 1000 Newtons (225 Lbf). Tie systems are also required to be designed using either working stress or limit states design methods. In CAN/CSA A370-84 the design wind load was calculated based on tributary areas. Many analytical studies have shown that the tie loads are not uniform, particularly for a flexible backup such as steel stud framing. The tributary areas for flexible backup are being factored up to account for the non uniform tie load distribution.

The working stress design methods from CAN/CSA A370-84 are being kept in the new standard alongside the new limit states design method. Each method will be discussed.

(i) Working Stress Design (WSD) Approach

Given that test results are available, the following is a list of the steps taken in the working stress design approach :

Step 1

Tests are conducted and the following data is recorded (Clause 12.1.3)

$\bar{x}$ average of the ultimate strengths of at least five tests

S.D. standard deviation of the tests

Step 2

To account for the variability in the test results the average of the maximum strengths is reduced by the variability factor (ψ), (Clause 12.1.3)

$$\psi = 1 - \frac{1.5}{\bar{x}}\left[\frac{\Sigma(X-\bar{x})^2}{n-1}\right]^{1/2}$$

where

X is each individual test result

n is the number of individual tests

This factor can be rewritten as $\psi = 1 - \dfrac{1.5S.D.}{\bar{x}}$

The resulting characteristic strength is $R_{char} = \psi\bar{x} = \left[1 - \dfrac{1.5S.D.}{\bar{x}}\right]\bar{x}$

which is simply $R_{char} = \bar{x} - 1.5S.D.$

Step 3

The safe working load of a connector is determined by dividing the characteristic strength, determined in accordance with Clause 12.1.3, by the applicable factor of safety from Clause 8.4.3.1

$$R_w = \frac{R_{char}}{F.S.}$$

where

F.S. = 2.0 for material failure of the metal connector

F.S. = 4.0 for embedment failure or elastic buckling failure of the connector

Step 4

The design wind load and the load on the tie is to be determined in accordance with CAN3-S304-M92 as follows :

(a) For Non-Flexible Backup

$$S_w = p\,A_{trib}$$

where

A_{trib} - is the tributary area for the tie and

p - is the design wind pressure calculated in accordance with the NBC as follows

$$p = q\,C_e C_g C_p$$

where

q is the 1 in 10 hourly wind pressure for a given location

C_e is the exposure factor which accounts for changes in wind speed with height and terrain

C_g is the gust factor which is equal to 2.5 in cladding design

C_p is the external pressure coefficient averaged over the area considered.

(b) For Flexible Backup

S_w shall be

1) the greater of

$$S_w = 2pA_{trib}$$

with p determined as for a non-flexible backup, i.e., the design wind is doubled

OR

$$S_w = 0.4p\,h_{veneer}s_{hor}$$

where

h_{veneer} is the height of the veneer

s_{hor} is the horizontal tie spacing

i.e., the tie load is 40 % of the that on the vertical strip

OR

2) determined by detailed stiffness analysis before and after veneer cracking

Step 5
At the working load level the resistance should exceed the design wind load :

$$R_w \geq S_w$$

or in terms of the characteristic strength and factors of safety :

$$\frac{R_{char}}{F.S.} \geq S_w$$

(ii) Limit States Design (LSD)

In the limit states design approach the following general criterion governs :

Resistance $\geq$ Effect of Load

or in notation form

$$R \geq S$$

where the resistance is calculated by using the characteristic strength as determined previously and applying a resistance factor as follows :

$$R_w = \phi R_{char}$$

where

$\phi = 0.9$ for material failure of the metal components or the connector or

$\phi = 0.55$ for embedment failure or failure of the fasteners, or elastic buckling failure of the connector.

and where the effect of factored loads is calculated in accordance with CAN3-S304:

$$S = \alpha_D D + \lambda\psi(\alpha_L L + \alpha_Q Q + \alpha_T T)$$

where

D is the specified dead load
L is the specified live load
Q is the specified wind or earthquake load
T is the load due to temperature changes, shrinkage, or creep of component materials, or from differential settlement
α_D is load factor for dead loads (1.25)
α_L is the load factor for live loads (1.5)
α_Q is the load factor for wind and earthquake loads (1.5 for wind and 1.0 for earthquake)
α_T is the load factor for thermal and differential movement (1.25)
λ is the importance factor (1.0 for most structures)
ψ is the load combination factor (1.0 when only one of L or Q or T is considered, 0.70 when two are considered and 0.6 when all three are considered)

With a brick veneer wall, the dead load of the veneer is not applied to the tie connectors. Thus, where only the effect of the wind is being considered, the effect of the factored loads simplifies to :

$$S = \alpha_Q Q$$

the wind load, Q, is computed using the same conditions for flexible and non-flexible backup (as per Step 4 for WSD), therefore

$$Q = S_w \quad \text{and} \quad S = \alpha_Q S_w$$

and the criterion $R \geq S$ can be written as

$$R_{char} \geq \frac{\alpha_Q}{\phi} S_w$$

Table 2-5 compares the two design methods. As is shown the two methods give different factors of safety. This is intentional as the WSD will be phased out by the year 2000 and the lower factors of safety in the LSD method are thought to be more accurate due to the increased control and knowledge of material properties.

Table 2-5 - Comparison of WSD and LSD Methods for Masonry Connectors

	WSD	LSD
Governing Criterion	$R_{char} \geq F.S. S_w$	$R_{char} \geq \frac{\alpha_Q}{\phi} S_w$
	Factors	Factors
Tension Material Failure	F.S. = 2.0	$\phi = 0.9,\ \alpha_L = 1.5$
Embedment or Elastic Buckling	F.S. = 4.0	$\phi = 0.55,\ \alpha_L = 1.5$
	Design Criterion	Design Criterion
Tension Material Failure	$R_{char} \geq 2 S_w$	$R_{char} \geq 1.67 S_w$
Embedment or Elastic Buckling	$R_{char} \geq 4 S_w$	$R_{char} \geq 2.73 S_w$

2.6.2 Load Distribution on Masonry Ties

The BIA Technical Note 44B (March 1987) does not contain specific requirements for the strength of wall ties but it does stress a flaw in the previous methods of determining tie loadings.

> *"Estimating tie loads based on tributary area can lead to large errors, depending on the geometry and properties of the wall system"*

This statement has been shown to be correct in numerous analyses. The loads on masonry ties are not uniform and are dependent on numerous factors such as stiffness of the two wythes, stiffness of the ties, boundary conditions, length of the two wythes etc. The new CAN/CSA S304 Code accounts for this non-uniform loading by factoring up the load based on tributary area. Besides making the determination of loading empirical, this factoring makes it difficult to know the true factor of safety that the ties are designed for with either the WSD and the LSD methods.

2.6.3 Relevance of Structural Requirements to Retrofit Applications

In a retrofit situation the installation of supplementary ties may be necessary as there may be corroded ties, missing ties or ties of inadequate strength or stiffness. The purpose of the extra ties may be to provide an increased degree of safety against failure or it may be to decrease veneer deflections by attempting to stiffen the wall system. Regardless of the purpose there has been little research on retrofit ties or on the implications of installing ties in a remedial situation.

Prior to the CSA/CAN A370-93 code it was left to the judgment of the engineer to assess safety if repairs were undertaken. Section 11 of the new A370 code specifically deals with repair connectors. The main items in this section are as follows :

1) repair connectors must be made of a material that is acceptable for new construction, but Level III connectors (stainless steel) are recommended when existing masonry connectors have experienced a corrosion related problem.
2) maximum thickness of repair connectors is 2/3 the thickness of the mortar joint as in new construction, unless the use of greater

thickness is proven to still satisfy the structural requirements of the masonry
3) repair connector strength shall satisfy the requirements for new connectors by either WSD or LSD methods.
4) note that some repair connectors induce additional stresses in the wall as a result of their method of fastening and this should be considered.
5) detailed description of field testing requirements for repair anchors

This study attempts to determine whether feasible and safe remedial connectors are available. Also of importance are the structural implications of installing retrofit ties into an existing wall. Structural concerns are whether the ties are safe, whether the stiffness of the ties influence the wall behaviour and where remedial ties should be located.

2.6.4 Factors of Safety

As the code switches from working stress design methods to limit states design methods the question arises as to the relative safety of the old and the proposed design methods.

(i) Working Stress Design (WSD)

Two factors of safety are applied in this method. The first is the correction factor that accounts for the variability in the test series. The characteristic value, which is based on the sample mean (from 5 tests) and a 1.5 standard deviation correction, ensures that the strength of any random tie connection will have a 93.32 % probability of being greater than the characteristic value. Secondly this characteristic strength is divided by 4 (for embedment or buckling failures) to obtain a safe working load. This is different from the previous CAN/CSA A370 standard which used a F.S. = 3.75 for a tributary area < 0.5 m^2, and a F.S. = 5 for a tributary area > 0.5 m^2. In the revision only one factor of safety is used, presumeably because the maximum allowable tie spacing is set at 800 x 600 which is a tributary area of 0.48 m^2. To determine how safe these ties are, confidence levels can be calculated. The normal deviate is how many standard deviations a value is from the mean.

$$z = \frac{x - \mu}{S.D.}$$

where

z is the normal deviate
x is a value
μ is the mean of the population which we have approximated with the mean of the sample
$S.D.$ is the standard deviation of the tests on the sample

For a particular normal deviate there are equivalent levels of confidence that any random choice from the population will be above that value. Table 2-6 lists normal deviates and the corresponding levels of confidence.

Table 2-6 Normal Deviates and Equivalent Levels of Confidence

Normal Deviate (Number of Standard Deviations)	Level of Confidence %	Number of ties below z in 10000
1	84.13	1587
1.5	93.32	668
2	97.72	228
3	99.86	14
4	99.997	0.3

In the case of masonry connectors the normal deviate is obtained by considering the safe working load as the value x and determining how many standard deviations it is away from the mean.

$$z = \frac{R_w - \bar{x}}{S.D.} \quad \text{or} \quad z = \frac{\frac{R_{char}}{F.S.} - \bar{x}}{S.D.}$$

Table 2-7 shows the resulting overall factor of safety for some typical examples of means and standard deviations. Also given is the normal deviate and the failure rate per 10000 ties for each of these cases. The definition of failure rate per 10000 ties is that number of ties out of every 10000 will have ultimate strengths below the safe working load or the factored characteristic strength.

Table 2-7 Example Calculations of Factors of Safety and Normal Deviates (WSD)

$\bar{x}$ (1)	$S.D.$	Coefficient of Variation %	R_{char}	$\frac{R_{char}}{F.S.}$ (2)	Overall Factor of Safety (1)÷(2)	Normal Deviate z	Number of Failures in 10000
1.5	0.075	5	1.388	0.347	4.29	15.37	< 0.3
	0.15	10	1.275	0.319	4.70	7.87	< 0.3
	0.5	33	0.750	0.188	7.98	2.624	43
2.0	0.1	5	1.850	0.463	4.32	15.37	< 0.3
	0.2	10	1.700	0.425	4.71	7.87	< 0.3
	0.67	33	1.000	0.250	8.00	2.624	43

Note : here F.S. = 4.0

This table stresses the influence of variability within a test series. The characteristic strength decreases considerably as the standard deviation increases. The characteristic strength has a 98% probability of being the mean of the entire population from which the sample of 5 tests were taken. Another way of looking at the characteristic strength is that there is only a 6.68% probability that a random tie will have a maximum strength below this characteristic value. When using confidence intervals normal deviates greater than 5 are not a good indicator and therefore the failure rate has been stated to be less than 0.3 failures per 10000 which is the failure rate for a normal deviate of 4.

The above approach does not address the loading side of the design criterion. In WSD the load is an expected load not a factored load. The method of calculating 1 in 10 wind design loads implies that each tie could experience this design load once in every 10 years. With flexible backup it has been found that the loadings on the ties are somewhat unpredictable and non-uniform (i.e. not directly related to tributary areas). To account for this the CAN/CSA S304 is requiring that, for flexible backup the tributary area and thus the load be either doubled or the tie load must be taken to be equal to 40% of that on the vertical design strip.

The WSD approach to incorporating safety makes no explicit reference to workmanship and installation practices. With masonry ties these two factors are very significant and highly variable. Nevertheless, considering the reserve in strength implicit in the WSD method it would seem that the resulting designs are on the high side.

Limit States Design Method (LSD)

In this method three adjustments or factors for safety are involved. The first is the variability factor implicit in choosing to modify the mean test value by 1.5 Standard Deviations to evaluate the characteristic value. The characteristic strength is computed in the same way as in the working stress design method. The second is the resistance factor (ϕ) which accounts for the fact that the actual strength may be less than anticipated due to variability of materials, dimensions, and workmanship. The third is the multiplier that is applied to the load; this load factor accounts for the possibility that loads larger than the specified service load may act on the structure.

Illustrative confidence levels can be calculated in the same manner as for the WSD method. Table 2-8 shows the resulting overall factor of safety for some typical examples of means and standard deviations. Also given is the normal deviate and the anticipated failure rate per 10000 ties for each of these cases.

Table 2-8 Example Calculations of Factors of Safety and Normal Deviates (LSD)

$\bar{x}$ (1)	$S.D.$	Coefficient of Variation %	R_{char}	$R_{char}\frac{\phi}{\alpha_Q}$ (2)	Overall Factor of Safety (1)÷(2)	Normal Deviate z	Number of Failures in 10000
1.5	0.075	5	1.388	0.509	2.95	13.21	< 0.3
	0.15	10	1.275	0.468	3.21	6.88	< 0.3
	0.5	33	0.750	0.275	5.45	2.45	71
2.0	0.1	5	1.850	0.678	2.95	13.21	< 0.3
	0.2	10	1.700	0.623	3.21	6.88	< 0.3
	0.67	33	1.000	0.367	5.45	2.45	71

The limit states design method allows the designer to use a higher factored resistance value for a connector and thus permits a smaller overall degree of safety than with the WSD approach.

2.6.5 Structural Serviceability

In the BIA Technical Note 44B (March, 1987) some guidelines regarding serviceability are outlined. To control deflection one of the suggested measures is to have stiff ties.

Stiff ties are defined as those having a maximum displacement of less than 1.2 mm (0.05 in) under an axial load of 0.45 kN (100 Lbf) in either tension or compression. This implies a minimum stiffness value of 375 N/mm (2000 Lbf/in).

The BIA Technical Note 28B Revised II (February, 1987) deals specifically with brick veneer steel stud walls. The recommendations are :

1) corrugated ties should not be used
2) ties should be two-piece adjustable ties with 50 mm embedment into the brick wythe
3) ties should be hot-dipped galvanized
4) ties should not have mechanical play in excess of 1.2mm (0.05 in)
5) ties should not deform in excess of 1.2mm (0.05 in) for 0.45 kN (100 Lbf) applied load in tension or compression.

In the 1993 draft version of the CAN/CSA A370 Connectors for Masonry Standard the very specific serviceability requirements follow the BIA guidelines closely. The requirements may be separated into 3 categories (Clause 8.3) :

1) Rigidity Requirements

   A masonry connector should have a rigidity compatible with the connected members and should be designed so that the resulting movements are acceptable.
2) Tie Displacement and Free Play

   The total free play of multi-component ties, including any free play between a tie component and the structural backing, shall not exceed 1.0 mm

   When tested under a compressive or tensile load of 0.45 kN, the sum of the displacement and free play of the tie shall not be more than 2.0 mm. Displacement includes all secondary deformations of the structural backing. Secondary deformations include fastener slippage, flange rotation, bending, and compression of loadbearing insulation or sheathing. Displacement does not include the primary deflection of the structural backing (i.e. bending of steel stud wall)
3) Differential Movement

   Consideration should be given to the effects of short and long term differential movements due to elastic deformations, creep, shrinkage, moisture changes and temperature changes.

The following reason for these serviceability criteria is given in Appendix A A1.2(e) in the CAN/CSA A370-93 standard;

"The total movement in the connector plus the deflection of the structural backing should be controlled to limit cracking of the veneer, which might lead to water penetration or structural weakness."

Characteristic values for the initial stiffness and free play are determined in the same way as for strength. In notational form this CAN/CSA A370 code criterion is :

$$\Delta_{char} \leq 2mm$$

where

$$\Delta_{char} = \psi \Delta_{0.45}$$

ψ is the factor that accounts for the variability within the tests

$\Delta_{0.45}$ is the average measured displacement at 0.45 kN of load including free play from testing

As before, the factor ψ is given by :

$$\psi = 1 - \frac{1.5}{\Delta_{0.45}}\left[\frac{\Sigma\left(\delta - \Delta_{0.45}\right)^2}{n-1}\right]^{1/2}$$

where

δ	is an individual test result
n	number of individual tests

which reduces to $\psi = 1 - \dfrac{1.5S.D.}{\Delta_{0.45}}$

The characteristic displacement can be expressed as :

$$\Delta_{char} = \Delta_{0.45} - 1.5S.D.$$

Therefore the serviceability criterion is :

$$\boxed{\Delta_{0.45} - 1.5S.D. \leq 2mm}$$

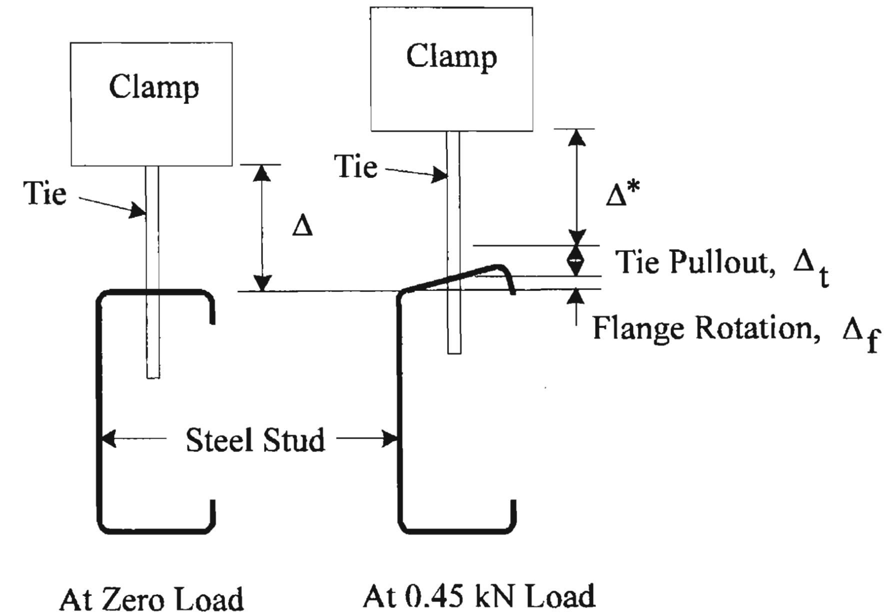
Basis For Serviceability Limits

The specification for ties of a formal serviceability limit state is both timely and significant. Although the limits imposed in CSA/CAN A370 seem reasonable, little justification for the choice of either 0.45 kN or 2 mm has been provided. The choice of 0.45 kN is however consistent with BIA guidelines.

To demonstrate the relevance of the 0.45 kN load with regard to the likely loading on a building, consider a representative building in Toronto. Using the methods discussed in section 2.6.1, the design suction pressure (worst case) for a 64 to 85 metre high building is 1.463 kPa (30.6 psf). This is based on a 1 in 10 year recurrence interval as required by the NBCC for the design of cladding. For a tie spacing of 400 x 400 mm the tributary area is 0.16 m^2. Because the backup is considered to be flexible, the tributary area must be doubled and this results in a tie load of 0.468 kN (105 Lbf). Therefore a wind load of 0.45 kN will occur about once every 10 years on a 20 to 30 storey building in Toronto. This is consistent with NBCC requirements for the design of cladding.

2.6.6 Application of Serviceability Requirements to BV/SS Walls

Because of the flexibility of not only the individual stud but also the steel stud backup system, the issue of displacement and associated serviceability limit state is especially important. It should be noted that the $\Delta_{0.45} - 1.5S.D. \leq 2mm$ design criterion only involves secondary deformations. Primary deformations are the lateral deformations due to lateral load (wind) and these are not included in this limit state. In fact the limit state for primary deformations is $\Delta_w \geq \frac{L}{720}$, which, depending on the model used, may or may not incorporate secondary displacements. Secondary deformations include the displacement of the tie, deformation of the sheathing and the rotation of the flange. Figure 2-14 illustrates the measurement of secondary displacements.



$$\Delta_{0.45\text{ kN}} = (\Delta^* - \Delta) + \Delta_t + \Delta_f$$

Figure 2-14 --- Deformations of Tie Connection During Testing

As testing is the vehicle for measuring the performance of the ties, whether new or retrofit, the test setup must accurately model the situation in the wall. Primary deflections can be measured. Secondary deformations should also be measured but they are not independent of the test set-up. It is, however, difficult to provide a test setup that accurately resembles the in-wall condition with regard to the restraint provided by the sheathings. For ties that mount onto the web of the stud the secondary deformations will be much less. With ties that mount onto the flange of the steel stud, as with most remedial ties, the flange rotation may be substantial.

2.6.7 Relevance of Serviceability Requirements to Retrofit Applications

The latest CSA Standard A370-94 clearly indicates that the serviceability requirements should be met for both new and retrofit connectors. To be economical and feasible, retrofit ties must connect onto one or both flanges of the steel stud thus increasing the importance of secondary deformations in meeting the serviceability criteria.

In retrofit situations it is debatable whether it is always necessary, or even desirable, for the retrofit tie to satisfy all the serviceability criteria. For example, supplementary ties may only be needed for the purpose of safety, i.e., structural integrity. In which case it may be preferable to use a tie with some ability to redistribute load. Also note that in retrofit situations the serviceability criteria for stiffness and free-play are largely independent of the spacing of the ties. In retrofit situations the cost and installation of the ties is small relative to other repair costs. Therefore a closer tie spacing may be an economical means of meeting these serviceability criteria.

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3. EXPERIMENTAL TEST PROGRAM

3.1 Description of Test Program

In the test program, 4 different types of retrofit tie and 4 different gauges of steel stud as well as a number of different test methods and setups were examined. A total of 435 individual tests were conducted. To ensure a reliable average, five tests were conducted in each test series for each tie. In a few of the test series the results of only four tests have been presented due to difficulties with one test.

The following code was devised to describe each test conducted:

TIE TYPE - SERIES NAME - GAUGE - TEST NUMBER (e.g. Test HE-2-20-1)

Four different retrofit ties and a datum situation were considered. They are described in detail in section 3.2 with the codes for the various ties as follows :

RD Rigid Datum
HE Helifix Exterior Tie
DE Dur-O-Wal Exterior Tie
HI Helifix Interior Tie
DI Dur-O-Wal Interior Tie

Table 3-1 outlines how this code is used to identify each test series. This table also indicates the number of tests conducted in each test series for each tie. The test setup, loading type and cyclic pre-conditioning are described in detail in section 3.3

Table 3-1 -- Outline of Test Program

TEST SERIES	TEST SETUP	TEST TYPE	Number of Tests Conducted					Number Done
1a	Isolation	Tension-Varying Gauge	**Gauge**	**16**	**18**	**20**	**21**	D
			HE - Helifix Exterior Tie	O	O	5	O	5
			DE - Dur-O-Wal Exterior Tie	O	O	5	O	5
			HI - Helifix Interior Tie	O	O	5	O	5
			DI - Dur-O-Wal Interior Tie	O	O	O	O	0
1b	Isolation	Compression	**Gauge**	**16**	**18**	**20**	**21**	
			HE - Helifix Exterior Tie	5	5	5	5	20
			DE - Dur-O-Wal Exterior Tie	O	O	3	O	3
			HI - Helifix Interior Tie	3	5	5	5	18
			DI - Dur-O-Wal Interior Tie	5	5	5	5	20
1c	Isolation	Cyclic Pre-Conditioning Tension to Failure	**Gauge**	**16**	**18**	**20**	**21**	
			HE - Helifix Exterior Tie	5	5	5	5	20
			DE - Dur-O-Wal Exterior Tie	5	5	5	5	20
			HI - Helifix Interior Tie	5	5	5	5	20
			DI - Dur-O-Wal Interior Tie	O	O	O	O	0
1d	Isolation	Cyclic Pre-Conditioning Compression to Failure	**Gauge**	**16**	**18**	**20**	**21**	
			HE - Helifix Exterior Tie	O	O	5	O	5
			DE - Dur-O-Wal Exterior Tie	O	O	O	O	0
			HI - Helifix Interior Tie	O	O	5	O	5
			DI - Dur-O-Wal Interior Tie	O	O	5	O	5
2	Beam	Tension Varying Gauge	**Gauge**	**16**	**18**	**20**	**21**	
			HE - Helifix Exterior Tie	5	5	5	5	20
			DE - Dur-O-Wal Exterior Tie	5	5	5	5	20
			HI - Helifix Interior Tie	5	5	5	5	20
			DI - Dur-O-Wal Interior Tie	O	5	5	5	15
3	Beam	Tension-Varying Attachment Location	**Position**	**10**	**20**	**30**		
			HE - Helifix Exterior Tie	5	5	5		15
			DE - Dur-O-Wal Exterior Tie	5	5	5		15
			HI - Helifix Interior Tie	5	5	5		15
			DI - Dur-O-Wal Interior Tie	O	O	O		0
4	Beam	Compression	**Gauge**	**16**	**18**	**20**	**21**	
			RD1 - Rigid Datum - 1 flange	O	O	1	1	2
			HE - Helifix Exterior Tie	5	5	5	5	20
			DE - Dur-O-Wal Exterior Tie	O	5	5	5	15
			HI - Helifix Interior Tie	4	5	5	4	18
			DI - Dur-O-Wal Interior Tie	4	5	5	5	19
5	Beam	Cyclic Pre-Conditioning Tension to Failure	**Gauge**	**16**	**18**	**20**	**21**	
			RD1 - Rigid Datum - 1 flange	2	2	3	3	10
			RD2 - Rigid Datum - 2 flange	O	O	3	3	6
			HE - Helifix Exterior Tie	O	5	5	5	15
			DE - Dur-O-Wal Exterior Tie	O	5	5	5	15
			HI - Helifix Interior Tie	O	5	5	5	15
			DI - Dur-O-Wal Interior Tie	O	O	O	O	0
6	Beam	Cyclic Pre-Conditioning Compression to Failure	**Gauge**	**16**	**18**	**20**	**21**	
			HE - Helifix Exterior Tie	O	O	5	O	5
			DE - Dur-O-Wal Exterior Tie	O	O	4	O	4
			HI - Helifix Interior Tie	O	O	5	O	5
			DI - Dur-O-Wal Interior Tie	O	5	5	5	15
			TOTAL NUMBER OF TESTS					**435**

O — NO TESTS WERE DONE

In Canada, the majority of buildings that were built in the 1980's used 20 gauge studs as indicated in an industry survey by H. Keller for CMHC in 1985[1]. This survey involved 160 respondents who indicated that the most common stud sizes were 25% - 92mm, 20 gauge, 22% - 152mm, 20 gauge and 22% - 152mm, 18 gauge. Presently 18 and 20 gauge are commonly used for new construction but 16 gauge studs are also used. The Brick Institute of America Technical Note 28B Revised II (Feb. 1987)[2] and the CMHC advisory document[3] recommend a minimum of 18 gauge stud. As some buildings were built with steel stud of 22, 24 and even 26 gauge it was desired to test the ties with light gauge stud. However, steel stud manufacturers no longer manufacture steel stud in these gauges. The lightest gauge of stud presently manufactured is 0.033" thick, approximately 21 gauge, and this is essentially an interior grade stud with a smaller flange width and indentations on the flange.

Four different gauges or thicknesses of steel stud were used in the test program. Table 3-2 describes key dimensions for the different steel studs. Figure 3-1 illustrates the dimensions that describe the steel stud.

Table 3-2 - Description of Steel Studs

Gauge	Thickness t inches	Thickness t mm	Overall Web Height, h mm	Overall Flange Width, w mm
16	0.060	1.52	92	41
18	0.048	1.22	92	41
20	0.036	0.91	92	41
21	0.033	0.88	92	31

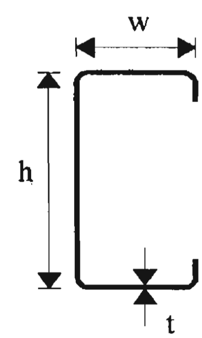


Figure 3-1 - Steel Stud Properties

3.2 Description of Tie Types in Test Series

3.2.1 Rigid Datum

In the testing of tie connectors in steel stud the deformations that result are due to tie pullout as well as deflection and flange rotation of the steel stud. The deformation due to each of these factors is important in determining the in-service performance of a tie system. In an attempt to isolate the deformation due to the steel stud and that due to tie pullout, a rigid datum connection was developed. The rigid datum connection consisted of a 6mm diameter threaded rod that was secured to one or two flanges with nuts as shown in Figure 3-2. The testing of this tie in tension, compression and under cyclic loading demonstrates how the steel stud itself deforms under loading for the boundary conditions in the test setup, i.e., there is no deformation due to the tie.

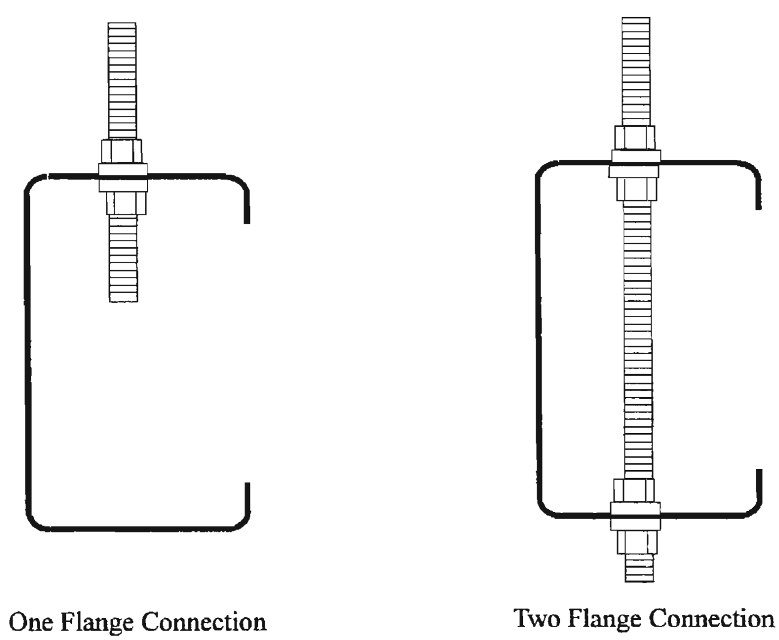


Figure 3-2 - Rigid Datum Tie Connection

3.2.2 Helifix Exterior Tie

This tie is installed from the exterior of the wall and engages the outer flange only. Figure 3-3 illustrates the exterior repair method using the Helifix tie. Table 3-3 describes the installation method used for the exterior Helifix tie. To maximize the performance of the tie connection different diameters of the pre-drilled hole are used as indicated in Figure 3-3.

3.2.3 Dur-O-Wal Exterior Tie

This tie is installed from the exterior and engages the outer flange only. Figure 3-4 illustrates the exterior repair using the Dur-O-Wal threaded lagbolt tie. Table 3-4 describes the installation method used for the exterior Dur-O-Wal tie.

3.2.4 Helifix Interior Tie

This tie is installed from the interior and engages both flanges of the steel stud. Figure 3-5 illustrates the interior repair using the Helifix Tie. Table 3-5 describes the installation method used for the interior Helifix tie.

3.2.5 Dur-O-Wal Interior Tie

This tie is installed from the interior and engages both flanges of the steel stud. Figure 3-6 illustrates the repair and outlines the repair procedure using the Dur-O-Wal epoxied rod interior tie. Table 3-6 describes the installation method used for the interior Dur-O-Wal tie.

FIGURE 3-3 --- Helifix Exterior Repair - HRT80 Dry Fix in SS, Polyester Resin in BV

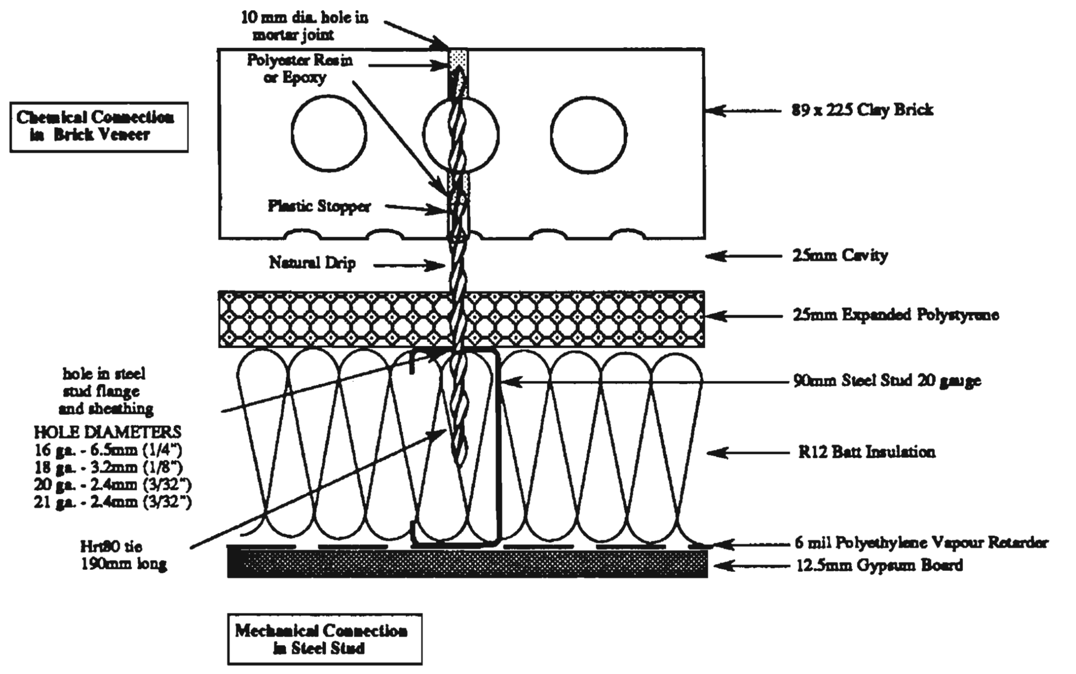


Table 3-3 --- Installation Procedure for Helifix Exterior Tie

STEP #	DESCRIPTION
1	Locate Stud
	Drill Through Brick Veneer at Mortar Joint - 10mm dia. masonry drill bit - hammer drill action
2	Drill Through Outer Steel Stud Flange - diameter as indicated in Figure 3-3 - rotary drill action only
3	Drive HRT80 tie into the outer flange of the steel stud
4	Inject Polyester Resin around the HRT80 tie
5	Repair Hole

FIGURE 3-4 --- Dur-O-Wal Exterior Repair - Threaded Bolt and Expansion Anchor

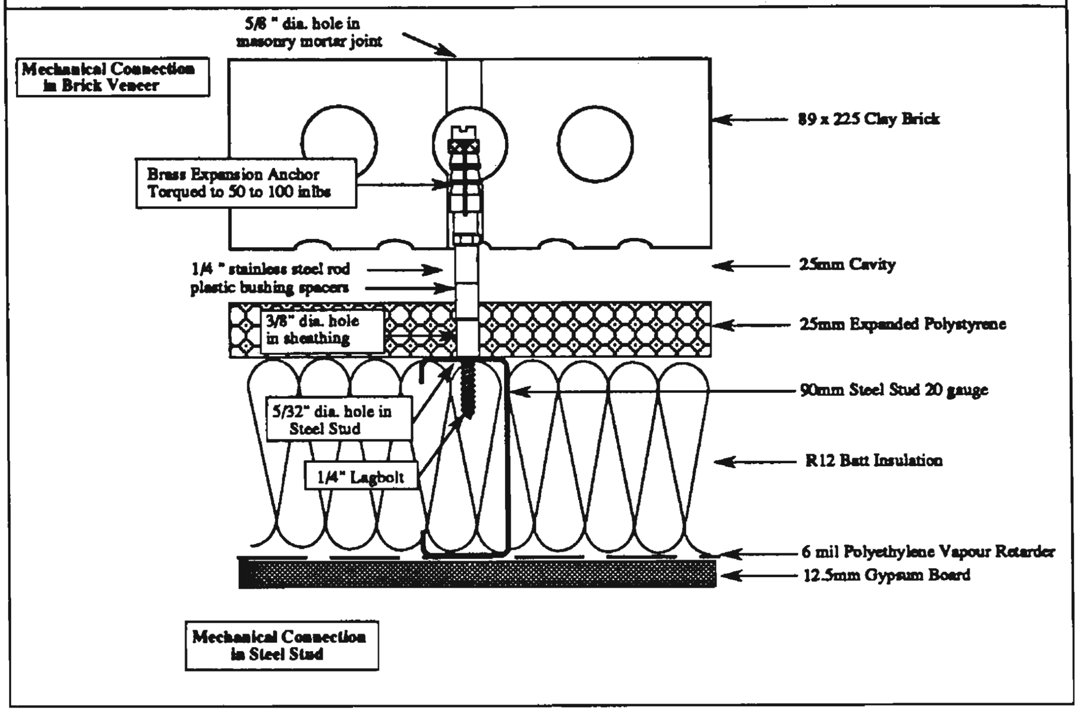


Table 3-4 ---Installation Procedure for Dur-O-Wal Exterior Tie

STEP #	DESCRIPTION
1	Locate Stud
2	Drill Through Brick Veneer Mortar Joint - 5/8" dia. masonry drill bit - rotary drill action
3	Drill Through Outer Flange of Steel Stud - 5/32" dia. steel bit
4	Blow debris out of hole
5	Slide bushings on the anchor rod
6	Apply silicone to the threads on the lag bolt
7	By hand, screw the lag bolt in and then torque to 25 to 50 in lbs
8	Install Expansion Anchor and torque to 50 to 100 in lbs
9	Repair Hole

FIGURE 3-5 --- Helifix Interior Repair - HRT80 Tie Dry Fix

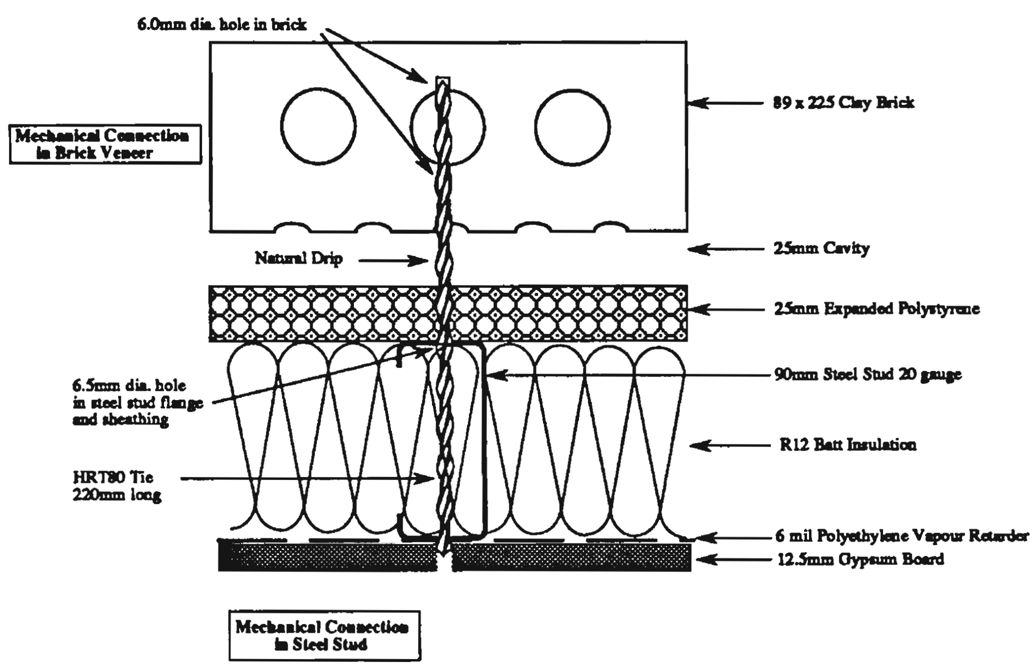


Table 3-5 --- Installation Procedure for Helifix Interior Tie

STEP #	DESCRIPTION
1	Locate Stud
2	Drill Through Steel Stud Flanges - 6.5mm dia. steel drill bit - rotary drill action only
3	Drill Into Brick Veneer - 6mm dia. masonry drill bit - hammer drill action - mark distance on drill bit
4	Drive HRT80 tie through the steel stud and into the brick
5	Repair Hole

FIGURE 3-6 --- Dur-O-Wal Interior Repair - Stainless Steel Rod and Sleeve with Epoxy

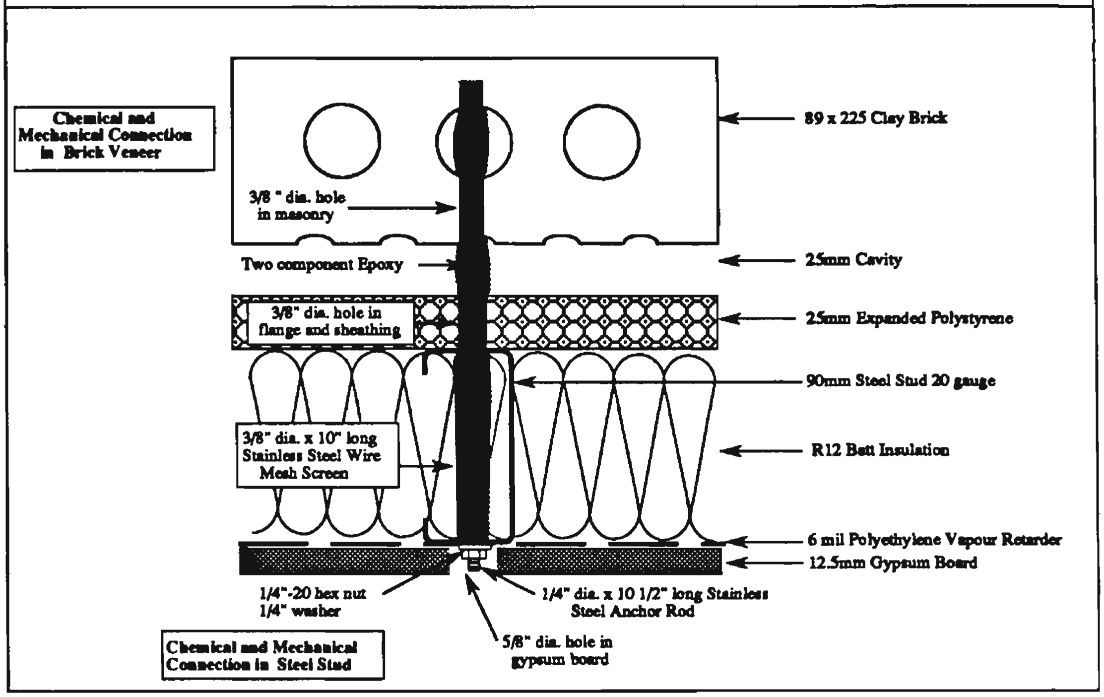


Table 3-6 --- Installation Procedure for Dur-O-Wal Interior Tie

STEP #	DESCRIPTION
1	Locate Stud
2	Drill Through Gypsum and Steel Stud Flanges - 5/8" dia. in gypsum, 3/8" dia. in flanges - rotary drill action only
3	Drill Into Brick Veneer - 3/8" dia. masonry drill bit - rotary drill action - mark distance on drill bit
4	Blow debris out of hole
5	Fill wire mesh with epoxy outside of the wall
6	Position filled wire mesh in the wall
7	Insert anchor rod into wire mesh by rotating slowly
8	Repair Hole

3.3 Test Setup and Procedure

There is no officially accepted standard test method to establish the overall performance of ties in steel studs. A test method was developed by Drysdale[4] at McMaster University in previous research for CMHC on new ties for steel stud applications. To facilitate comparison of this test program on retrofit ties with the previous program on new ties, the McMaster test method with some minor modifications was used in this test program. Figure 3-7 illustrates the test setup used in this test program while Figure 3-8 illustrates the McMaster test setup. The key differences, which were made to better model current practices, are as follows :

1) 400 mm span in BEG tests versus 450 mm span in McMaster tests
2) BEG tests were elevated off the loading table to allow beam movement during both tension and compression tests which more closely models the in-wall condition. The McMaster test setup for compression had the stud bearing directly on the loading table.
3) 41 mm flange width used in BEG tests versus 31 mm in the McMaster tests. BEG used the 41 mm flange width as this is what was commonly manufactured for wind load bearing studs.
4) 50 mm cavity width in BEG tests versus 25 mm cavity width in the McMaster tests, as this is more consistent with good practice.

The stud was effectively clamped at both ends. In the tests two displacements were measured, the beam displacement and the overall displacement. Figure 3-9 illustrates the displacements that were measured. The beam displacement was measured at the centerline of the stud and was subtracted from the total displacement to obtain the "tie" displacement. This method of measuring displacements was the same as that in the McMaster test program. The difference in length of the beam between the two test programs has a limited effect on the tie displacement, as only slightly greater flange rotation can be expected.

The test setup described above has been called the "beam" test. In addition to the beam test setup, the test program included tests that isolated the tie in the steel stud by eliminating both the beam displacement and most of the flange rotation. This test setup has been called the "isolation" test. Figure 3-10 illustrates this test setup. The intent of this test setup was to provide another test method to determine the influence of the test setup on the test results. Photos 3-1 and 3-2 show the setup for the "beam" and "isolation" tests respectively.

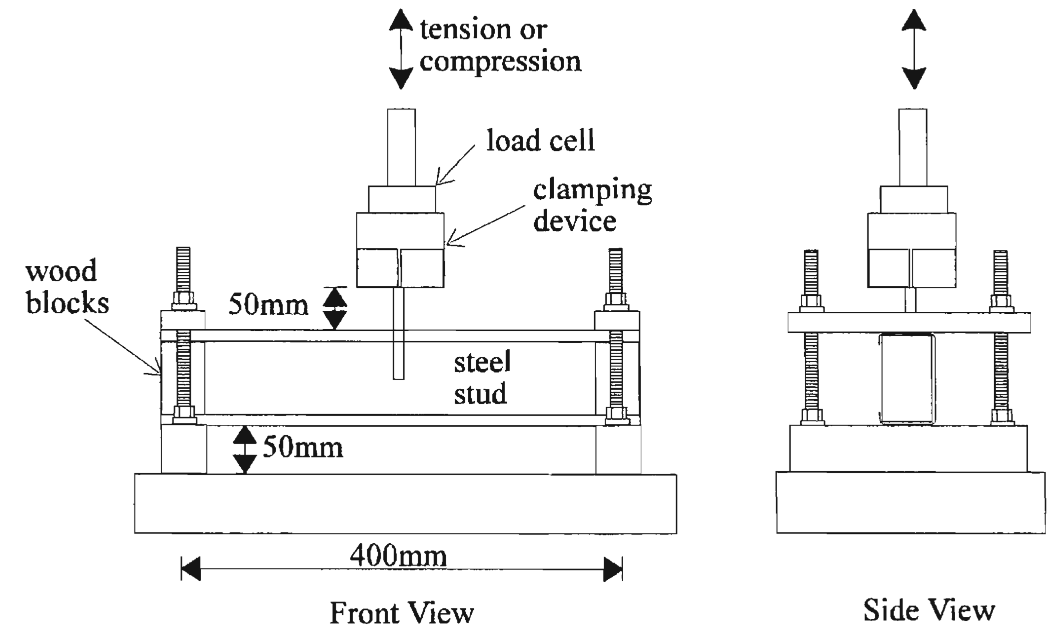


Figure 3-7 - Beam Test Setup, Waterloo Study

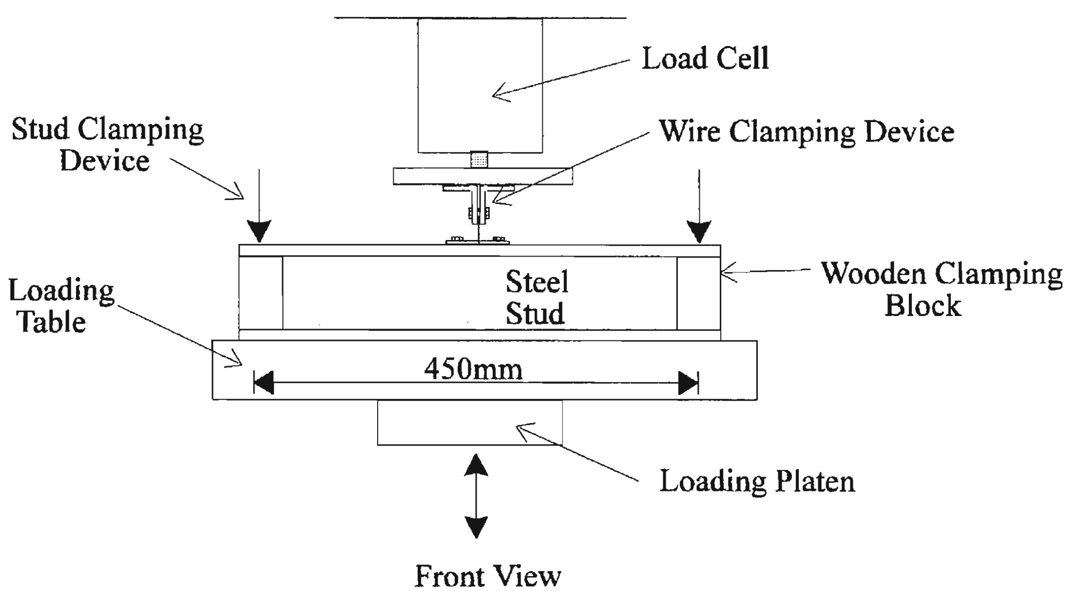


Figure 3-8 - Beam Test Setup, McMaster Study

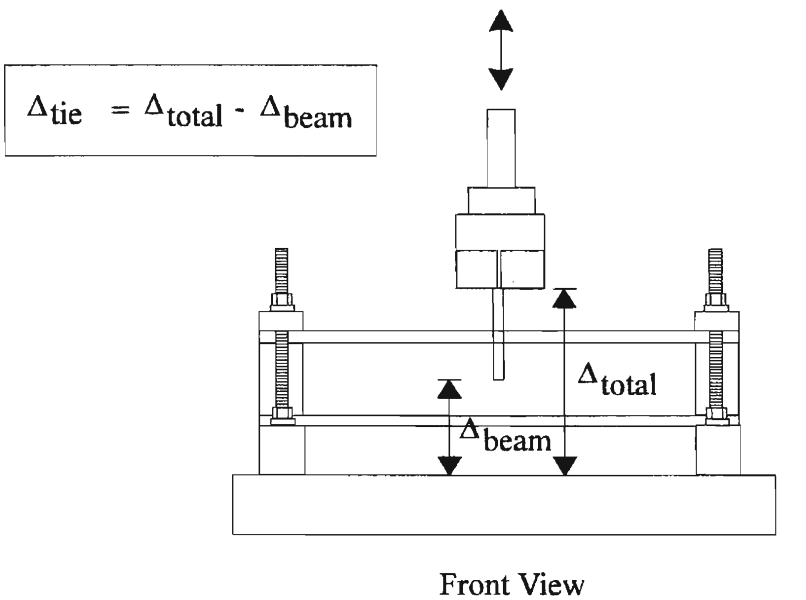


Figure 3-9 - Displacement Measurements

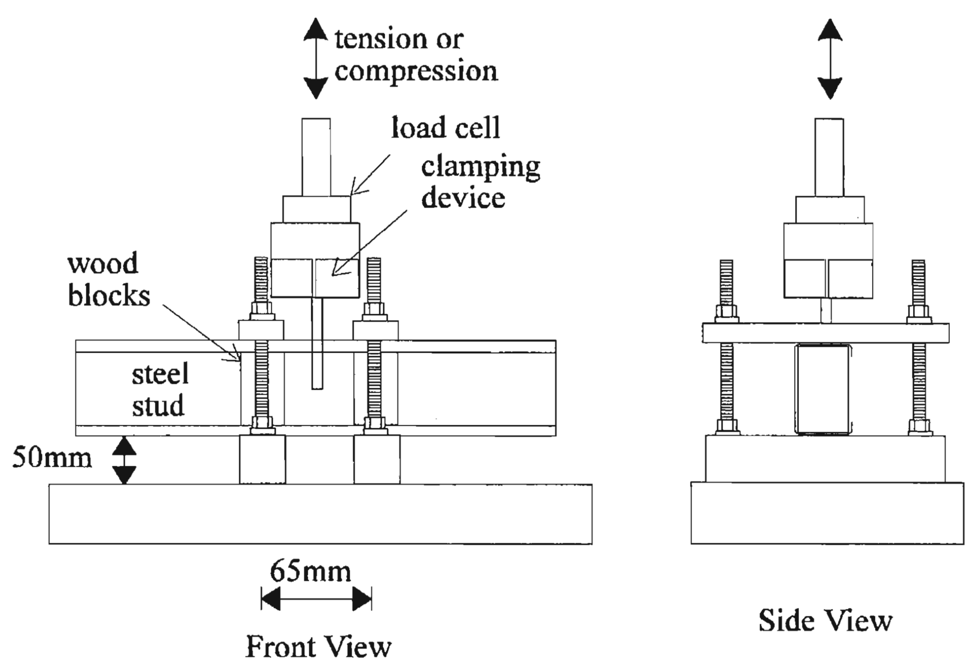


Figure 3-10 - Isolation Test Setup, Waterloo Study

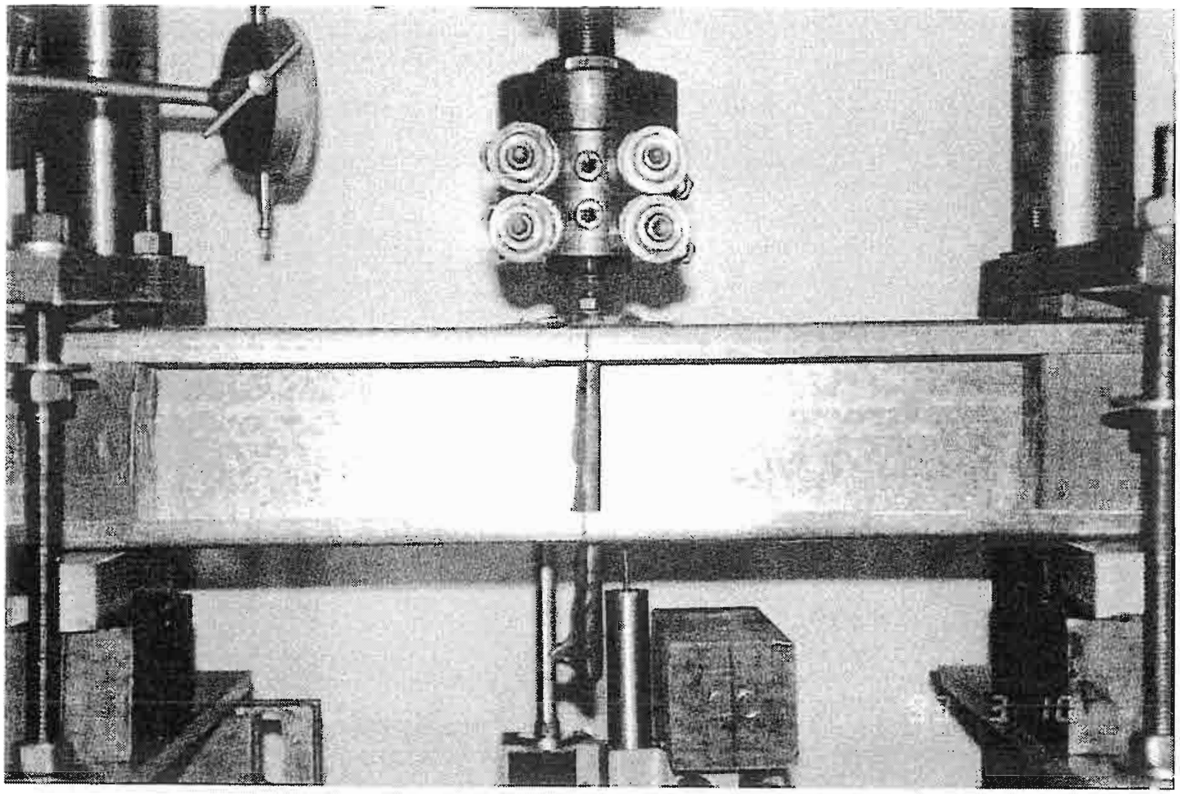


Photo 3-1 --- Typical Beam Test Setup

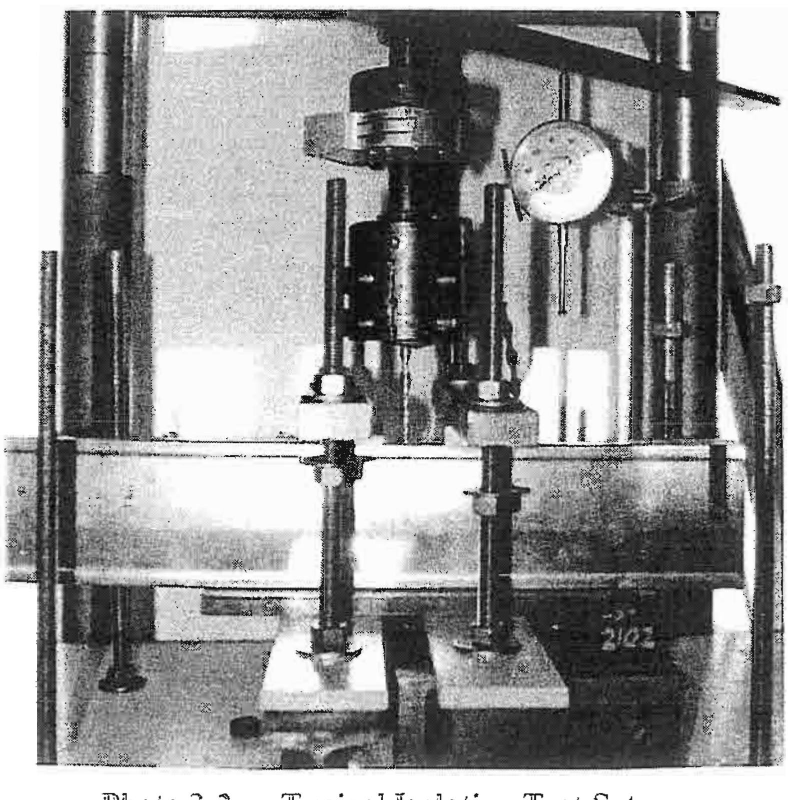


Photo 3-2 --- Typical Isolation Test Setup

Tests were conducted using a MTS electro-hydraulic test facility at the University of Waterloo. The ties were loaded under stroke control at a rate of approximately 6 mm/min. An X-Y plotter and a data acquisition system provided a continuous record of the loads and displacements.

Although cyclic pre-conditioning has not been routinely required in the testing of masonry ties, we are the opinion that it should be. Cyclic pre-conditioning is essential to assess tie performance under service load. For the pre-conditioned tests a sinusoidal loading with amplitudes of 0.15 kN (33 Lbf) tension and 0.15 kN (33 Lbf) compression was applied for 1000 cycles at 1.0 Hz, after which the specimen was failed under stroke controlled monotonic loading. The magnitude of this value was calculated considering the maximum allowable tie spacing permitted in the CSA/CAN A370-M93 Code, which gives a tributary area of 0.48 m^2. A design pressure of 1.0 kPa on this tributary area produces a design tie load of 0.48 kN. It was thought that 1000 cycles of loading from +1/3 to -1/3 of this service load value was representative of likely repeated loads.

During the load cycling stage both stud and overall displacement was recorded. The beam displacement at these low loads was negligible and the tie displacement was numerically close to the total displacement. The cyclic displacement was recorded early in the cycling stage, at 30 cycles, and then again at the end of cycling, i.e., at 1000 cycles. Figure 3-11 illustrates the loading path and resulting displacement during the load cycling.

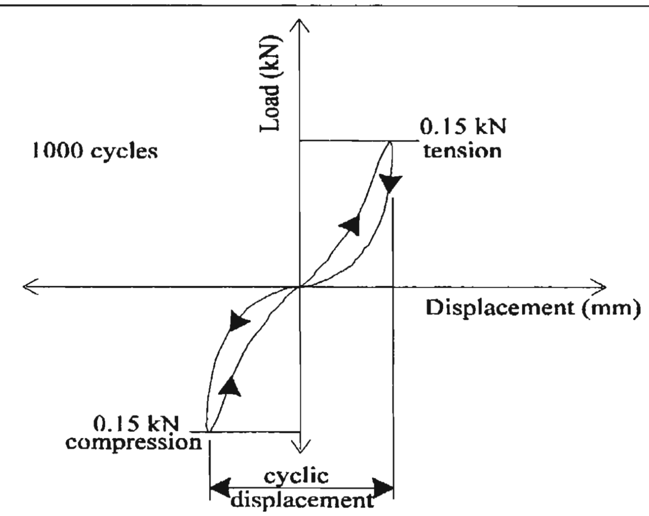


Figure 3-11 - Loading Path and Displacement During Cyclic Pre-Conditionng

As the ties are installed blind, the location of the tie in the flange may vary. The strength and stiffness of the connection, particularly for ties attaching to one flange, may depend on the attachment location. Tests to determine the influence of the location of the tie were conducted in Test Series 3. All four ties were tested but only with 20 gauge studs. The tests were limited to tension loading with no cyclic pre-conditioning. The ties were installed in the flange at 10, 20 and 30 mm from the web. Figure 3-12 illustrates the positioning of the ties at the different locations in the outer or inner flange.

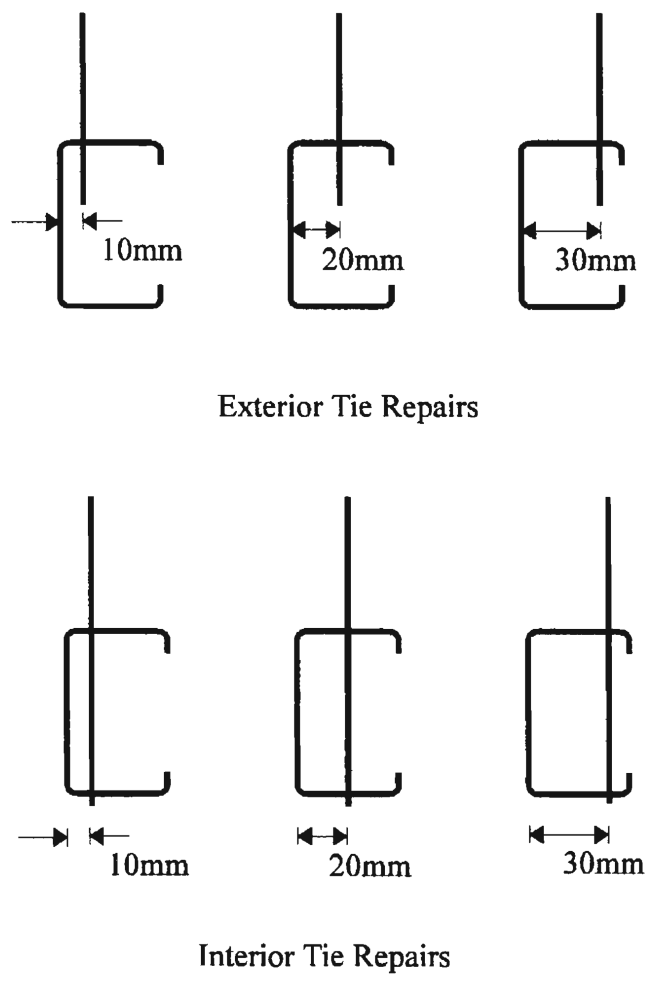


Figure 3-12 — Attachment Location of Ties in Test Series 3

References

[1] Keller, H. *Brick Veneer/Steel Stud Wall Design and Construction Practices in Canada,* Results of a 1986 Survey prepared by Suter Keller Inc. for CMHC, 1986

[2] Technical Notes on Brick Construction, *Brick Veneer Steel Stud Panel Walls*, Brick Institute of America, Virginia, No. 28B, Revised Feb., 1987

[3] Drysdale, R.G., Suter, G.T., *Exterior Wall Construction in High-Rise Buildings; Brick Veneer on Concrete Masonry or Steel Stud Wall Systems*, CMHC, Ottawa, Ont., 1991

[4] Drysdale, R.G., Wilson, M.J., *Behaviour of Brick Veneer/Steel Stud Tie Systems*, CMHC, Ottawa, Ont., 1989

4. TEST RESULTS

4.1 Organization of Test Results

The test results are presented separately for each tie type. The rigid datum tie is presented first as the performance of this connection depicts the behaviour of the stud alone.

For each test series summary tables have been developed. The detailed test results are collected in a separate volume, identified as Appendix A. These results, plus a more detailed discussion, are also available in Mark Postma's M.Sc. thesis available from the University of Waterloo. In this report the results are presented in an overall summary table for each tie. The summary tables contain all the test series tested for that tie. Relevant data such as the cyclic displacement (if applicable), the displacement at 0.45 kN, the load at 1 and 2 mm, the proportional limit load and its corresponding displacement and the maximum load and its corresponding displacement are listed in these summary tables. Figure 4-1 illustrates the other parameters used in presenting the test results. To quantify the initial stiffness of these connections the displacement at 0.45 kN was recorded together with the load at 1 and 2mm displacement. The proportional limit, P, was taken as the point where the linear relationship between load and deflection stopped and non-linear behaviour commenced. This point is sometimes difficult to ascertain and some variability is to be expected.

The performance of each tie is discussed under the following headings :

- Performance in Tension
- Performance in Compression
- Performance under Cyclic Loading
- Influence of Stud Gauge
- Influence of Attachment Location
- Comparison of Beam and Isolation Tests

The separation of the presentation of the results into these headings makes this chapter somewhat lengthy and a bit repetitive. However, this systematic structure is advantageous permitting a better focus on the test results.

Characteristic curves were obtained from the data acquisition system files by averaging the results from the five tests for each test series. Although an averaging technique was used, these curves are only representative curves of pullout response. The performance of the different ties, relative to the design performance requirements outlined in Chapter 2, is assessed in Chapter 5. Also in Chapter 5, a comparison of the performance of each retrofit tie is made and compared to results from tests conducted on ties used in new construction that were tested in the McMaster test program.

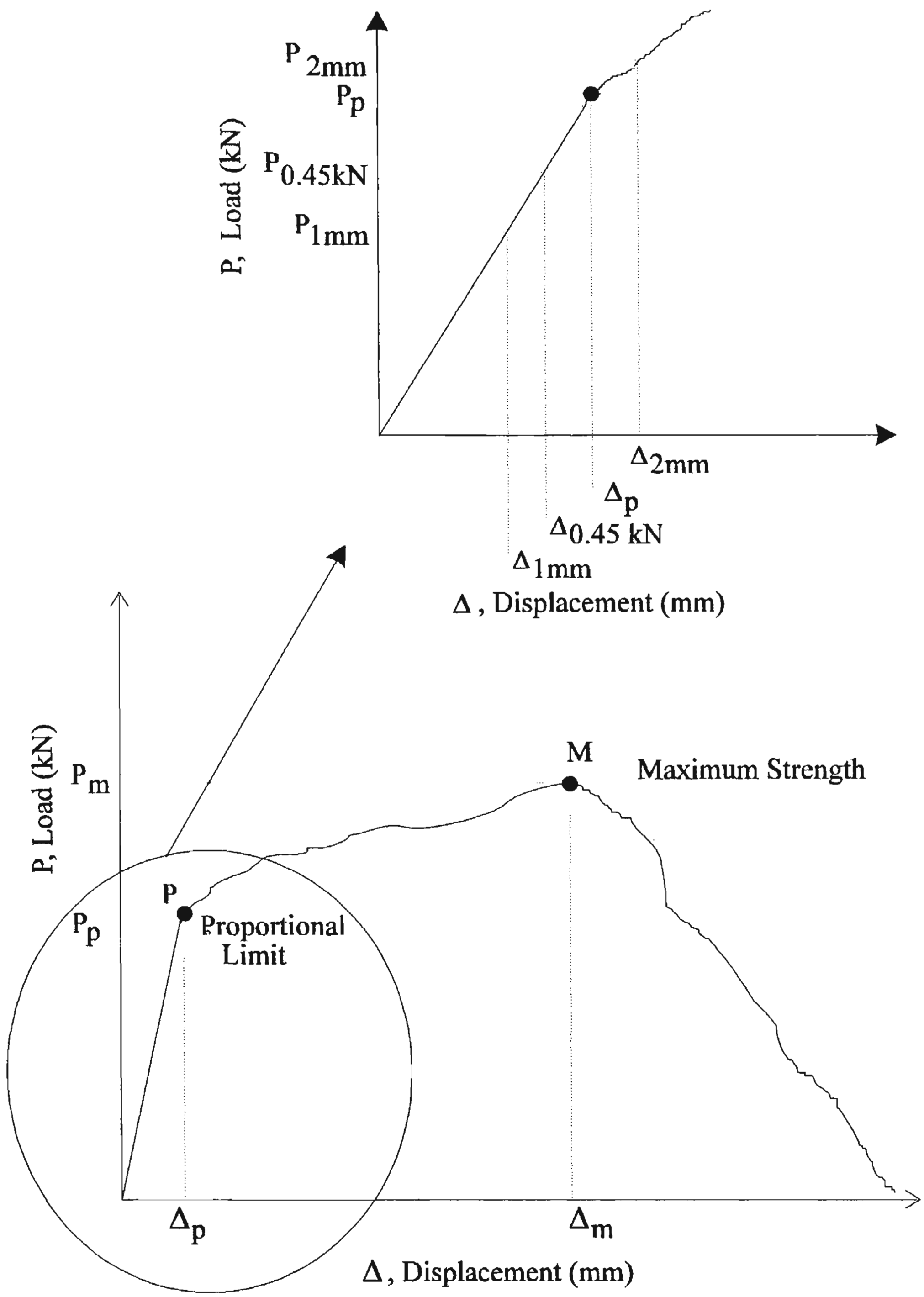


Figure 4-1 - Representative Load Versus Displacement Curve

4.2 Rigid Datum

Table 4-1 summarizes the results for all tests conducted on the Rigid Datum. The test results for the one flange rigid datum connection are denoted RD1 while those for the two flange rigid datum connection are denoted RD2. Maximum load values are not given in this table as the tests were never taken to maximum capacity but were stopped at approximately 5 kN. The objective of these tests was to establish the stiffness and deformational characteristics of the steel stud.

4.2.1 Performance in Tension

For the rigid datum tie, as shown in Figure 3-2, two cases were considered; namely a one flange connection and a two flange connection. The increase in stiffness with stud thickness is evident from the stiffness values listed in Table 4-1. Figure 4-2 illustrates the characteristic curves in tension for the one flange connection in all four gauges. As would be expected stiffness increases with increasing stud thickness.

For the two flange connection the characteristic curves in tension for 20 and 21 gauge are shown in Figure 4-3. The two flange connection is stiffer in tension than the one flange connection. This is also evident from the stiffness values in Table 4-1.

4.2.2 Performance in Compression

Tests in compression were only conducted on the one flange connection and only for 20 and 21 gauge studs. Figure 4-4 shows the characteristic curves in compression for 20 and 21 gauge superimposed on the tension characteristic curves. The initial stiffness is almost the same for tension and compression. At higher loads the response in tension remains linear for the tension tests but becomes non-linear for the compression tests. This occurs because the channel section stiffer in tension than in compression due to the configuration of the stud cross-section. Flange rotation and subsequent lateral torsional deformation of the stud occurs more readily in compression than in tension.

Table 4-1 - Results of Testing for the Rigid Datum

Load and Displacement Values

Tie Type	Test Series	n	Gauge	Cyclic Displacement mm	Load at 1 mm kN	Load at 2 mm kN	Displacement at 0.45 kN mm
RD1	5	2	16	0.11	1.51	2.55	0.25
	5	2	18	0.25	0.80	1.39	0.55
	5	3	20	0.6	0.42	0.75	1.1
	5	3	21	0.52	0.34	0.52	1.6
RD1	4	1	20	N.A.	0.36	0.65	1.3
	4	1	21	N.A.	0.37	0.54	1.4
RD2	5	3	20	0.16	1.01	1.66	0.35
	5	3	21	0.13	0.83	1.18	0.42

Stiffness Values

Tie Type	Test Series	n	Gauge	Stiffness based on Load at 1 mm N/mm	Stiffness based on Load at 2 mm N/mm	Stiffness based on Displacement at 0.45 kN N/mm
RD1	5	2	16	1510	1275	1800
	5	2	18	800	695	818
	5	3	20	420	375	409
	5	3	21	340	260	281
RD1	4	1	20	360	325	346
	4	1	21	370	270	321
RD2	5	3	20	1010	830	1286
	5	3	21	830	590	1071

RD1 - one flange rigid datum tie connection
RD2 - two flange rigid datum tie connection
Test Series 5 - tension, cyclic pre-conditioned
Test Series 4 - compression, not cyclic pre-conditioned
n - number of tests conducted
N.A. - not applicable to this test series

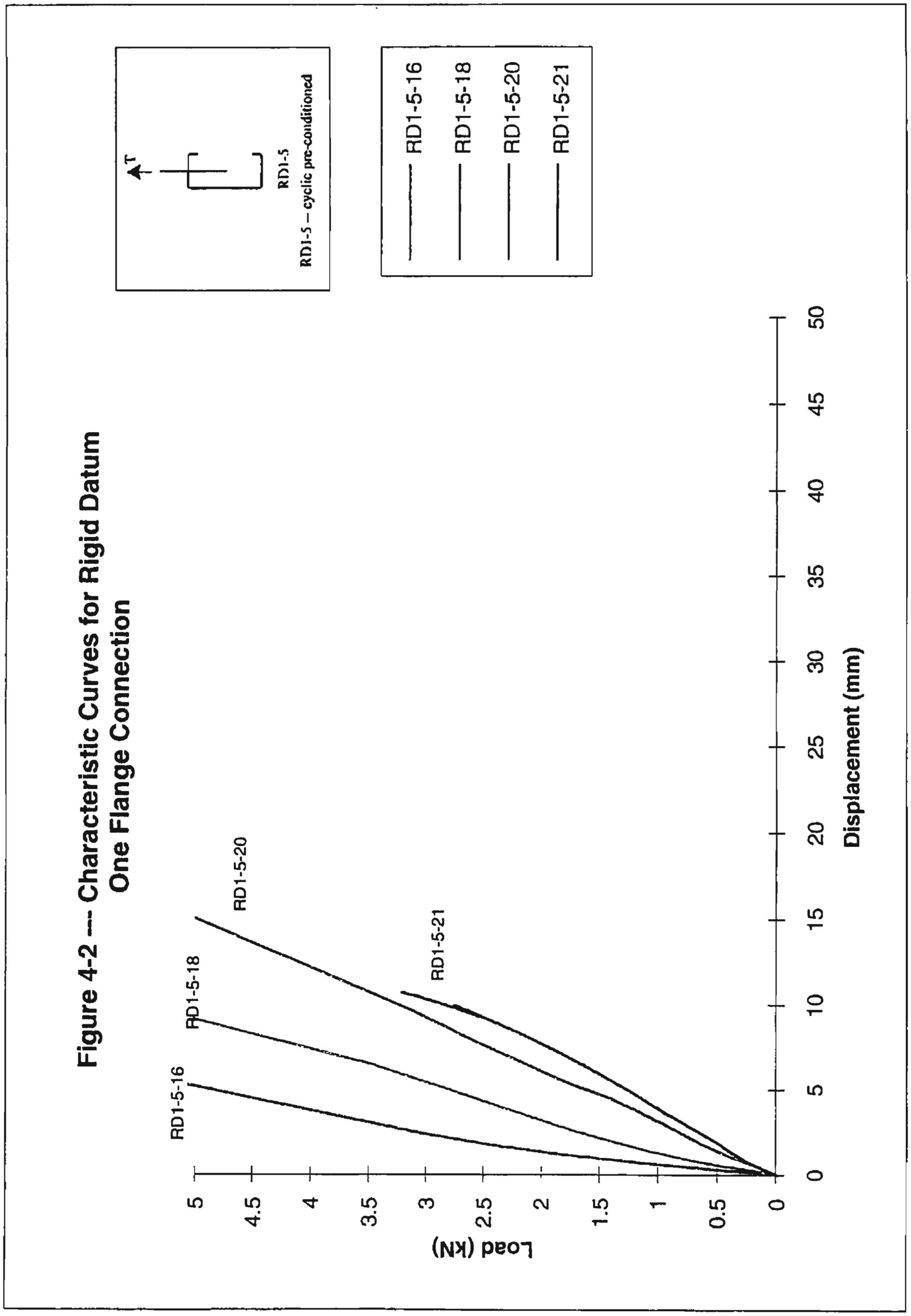


Figure 4-2 --- Characteristic Curves for Rigid Datum One Flange Connection

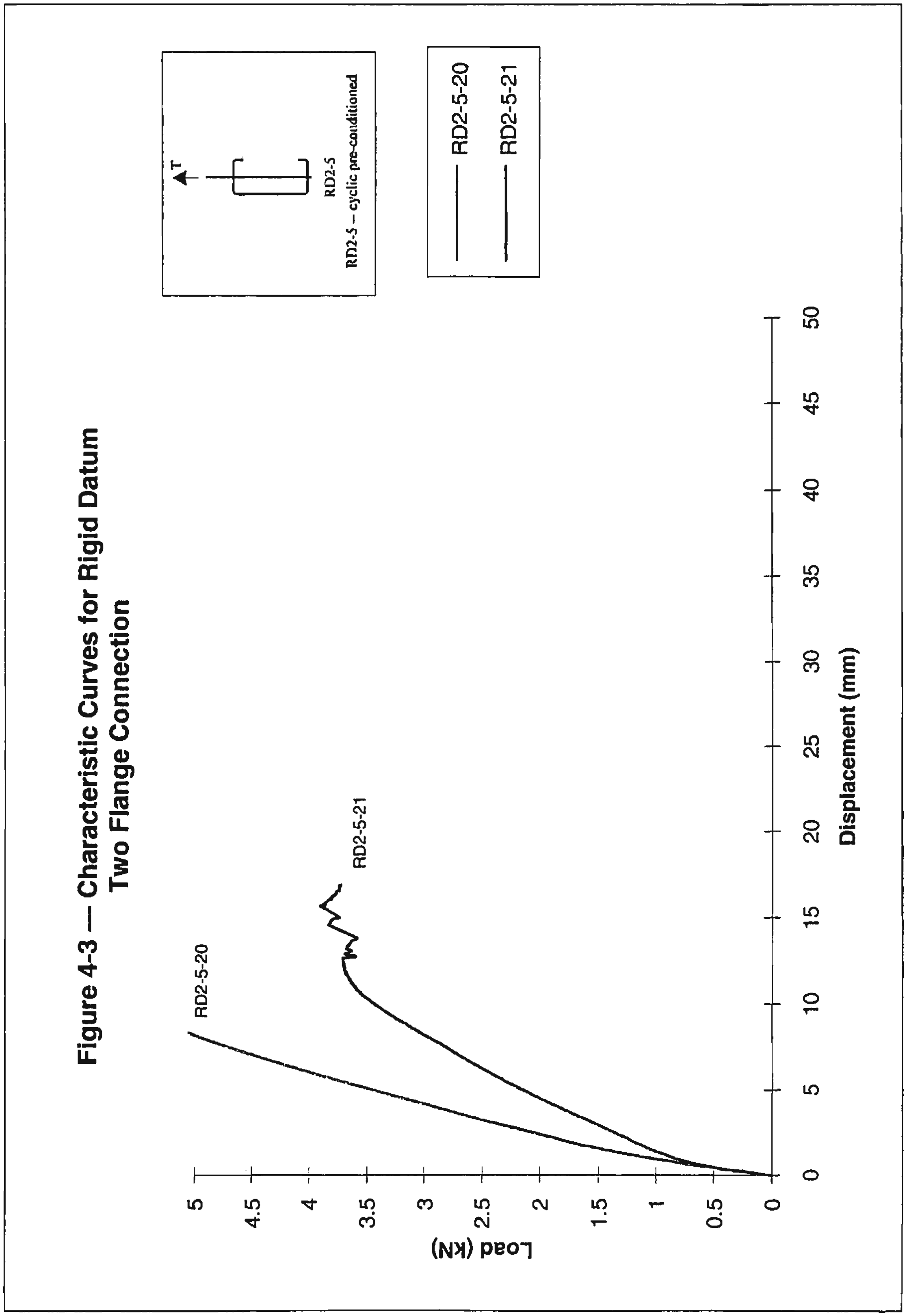


Figure 4-3 — Characteristic Curves for Rigid Datum Two Flange Connection

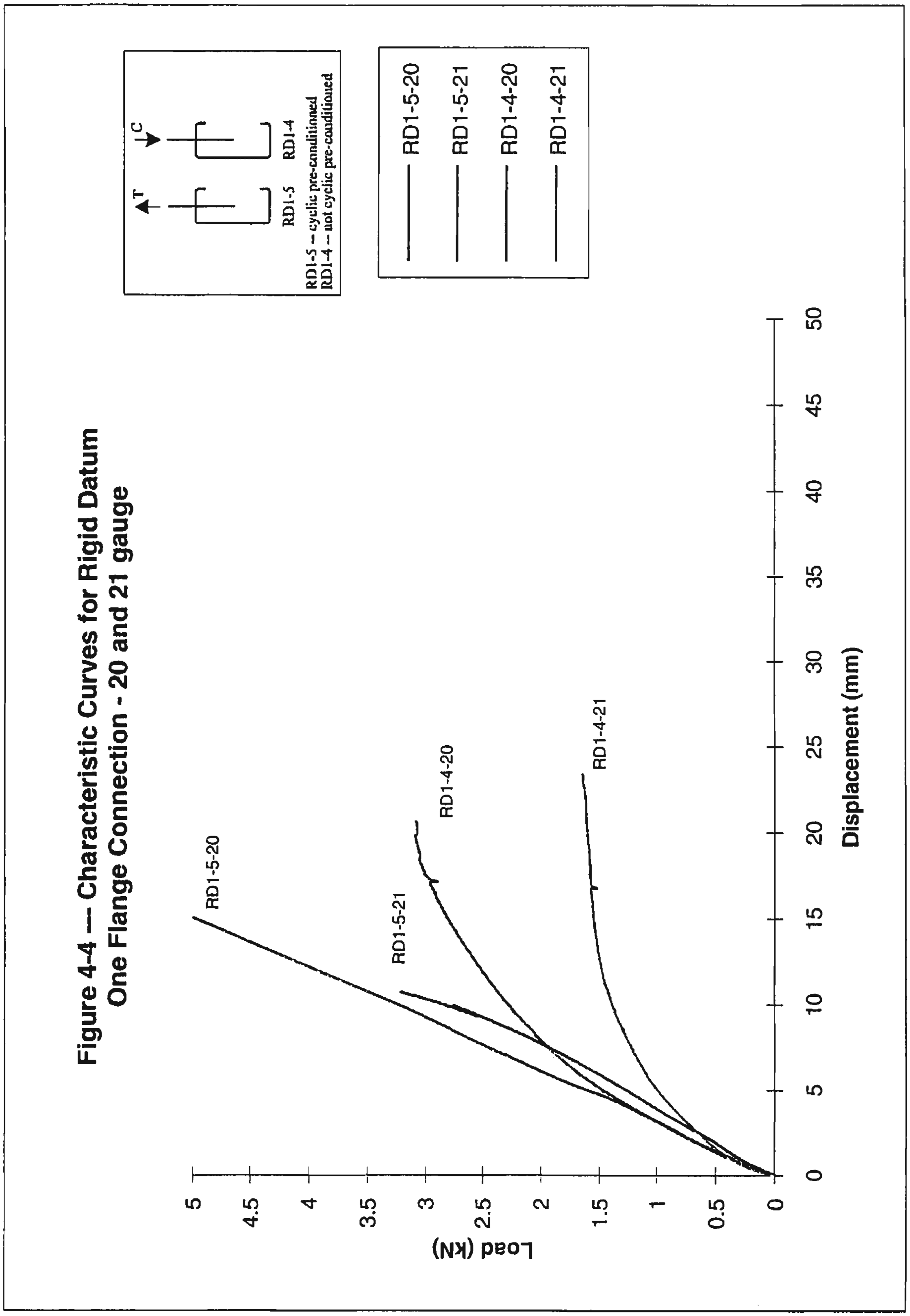


Figure 4-4 — Characteristic Curves for Rigid Datum One Flange Connection - 20 and 21 gauge

4.2.3 Performance Under Cyclic Loading

Cyclic pre-conditioning on the rigid datum connections was conducted to determine the magnitude of the cyclic displacements under the +/- 0.15 kN loading. Table 4-1 lists the amount of cyclic displacement for the one flange connection onto 16, 18, 20 and 21 gauge steel studs and for the two flange connection onto 20 and 21 gauge steel studs. The amount of cyclic displacement increased significantly with a decrease in stud thickness. The amount of cyclic displacement for the two flange connection was approximately one quarter of that for the one flange connection.

All tension tests with the rigid datum ties included cyclic pre-conditioning. As no tests were conducted without pre-conditioning the effect of pre-conditioning on the initial stiffness of this connection was not studied.

4.2.4 Influence of Stud Gauge

Figures 4-5 and 4-6 illustrate the characteristic curves for all tests conducted on rigid datum connections in 20 and 21 gauge steel stud respectively. It is evident, that for both the 20 and 21 gauge stud, the two flange connection is more stiff than the one flange connection. As well, Figures 4-5 and 4-6 illustrate that up to about 1.3 kN, the initial stiffness for the one flange connection is the same regardless of being loaded in tension or compression.

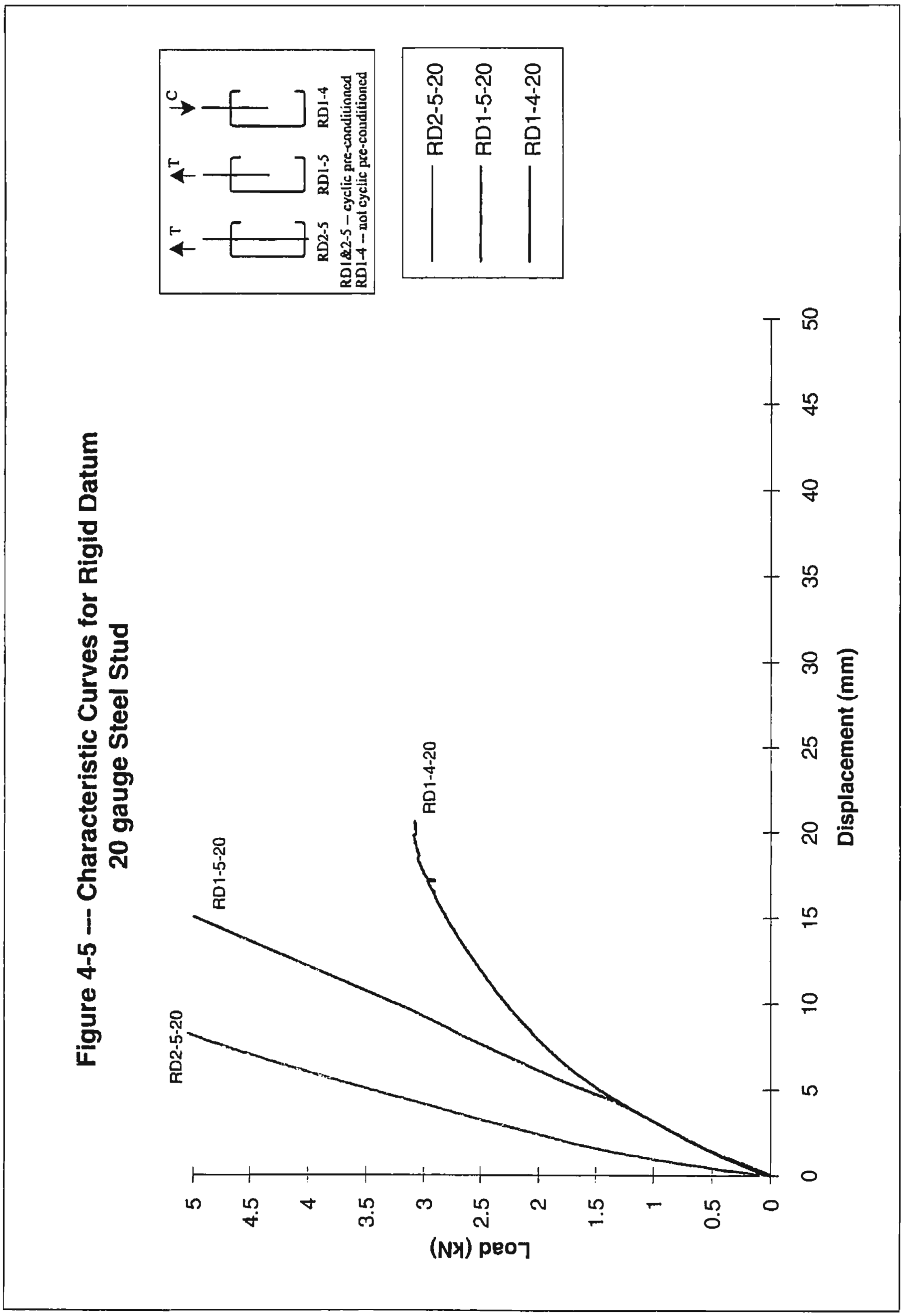


Figure 4-5 — Characteristic Curves for Rigid Datum 20 gauge Steel Stud

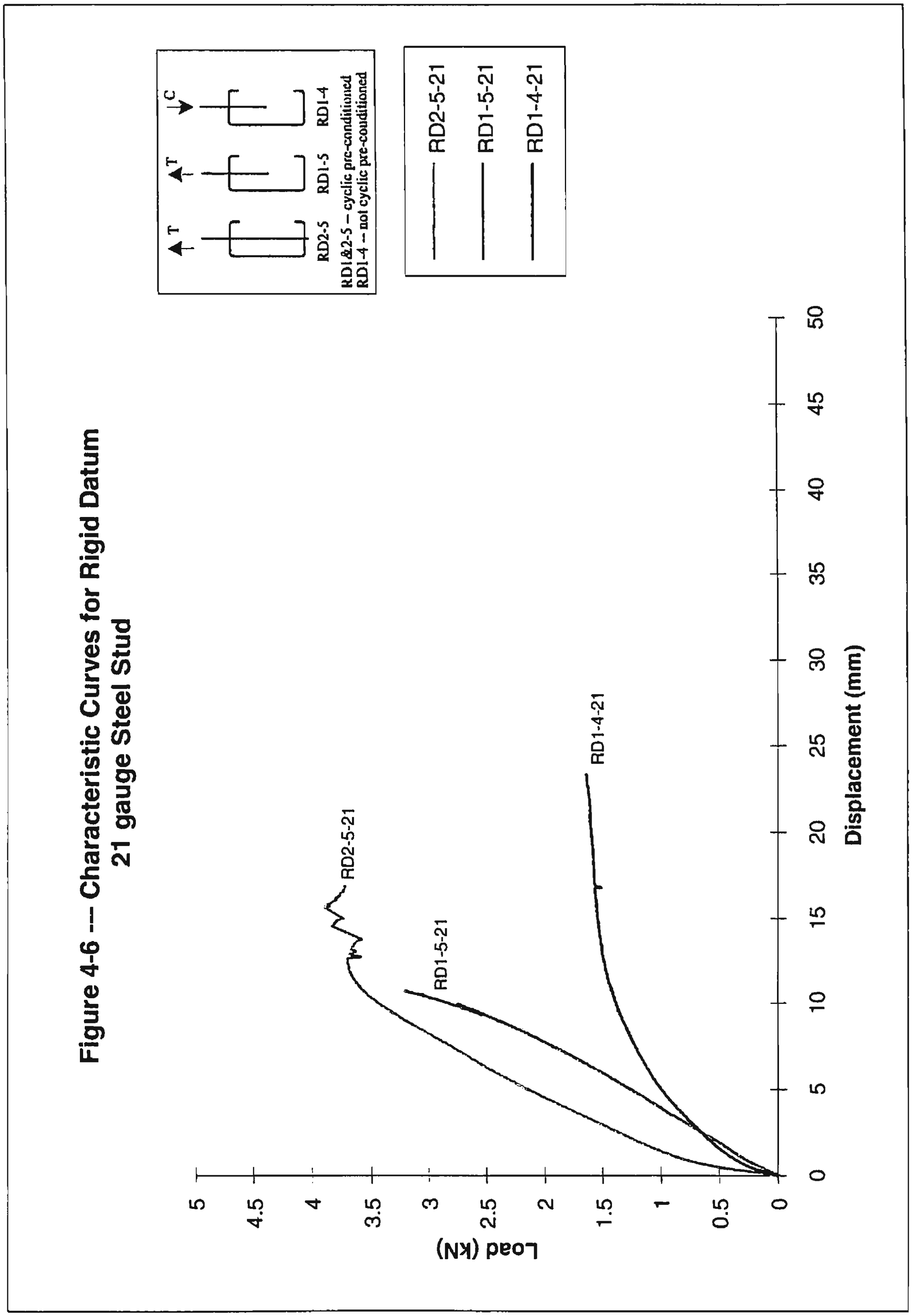


Figure 4-6 --- Characteristic Curves for Rigid Datum 21 gauge Steel Stud

4.3 Helifix Exterior Tie

Table 4-2 and 4-3 summarize the results from the tests conducted on the Helifix Exterior Tie. It is important to note that different pilot hole diameters were used for the different gauges of stud.

4.3.1 Performance in Tension (Series 2)

The performance of the Helifix Exterior tie in tension was characterized by an initially linear pullout response followed by a significant non-linear ductile pullout response. Figure 4-7, which shows the characteristic curves in tension for each of the four gauges, illustrates this ductility. The initial stiffness increased with steel stud thickness. The nature of failure was very different for the different gauges of stud. In both the 16 and 18 gauge tests the flanges bent and the tie unraveled and suffered damage but with little damage to the steel stud flange. In the tests on the 20 and 21 gauge studs, the tie itself was not deformed but failure occurred with tearing of the steel stud flange.

4.3.2 Performance in Compression (Series 4)

The performance of the Helifix Exterior Tie in compression is depicted in Figure 4-8. Initially response was linear followed by a flattening to a relatively constant strength. At approximately 10 mm the load spiked up to a maximum load. This spike is due to the tie hitting the other flange of the stud after 10mm of displacement. There was considerable variability in the magnitude of this peak and it should not be relied on for design. In Table 4-2 the maximum load listed is the load prior to this spike. Test Series HE-4A was conducted with the end of the tie positioned 25mm into the flange leaving almost 70mm of displacement before any spike would occur. The result is that the initial stiffness is essentially the same but the maximum load is significantly lower as there is no load spike. The tie was slightly stiffer in compression than it was in tension for all gauges of stud. Heavier gauge studs exhibited greater stiffness and higher maximum load.

Table 4-2 : Average Values for the Helifix Exterior Repair

Test Series	Gauge	Position	Pilot Hole	N	Cyclic Displ. * mm	Displ. at 0.45kN mm	Load at 1.0 mm kN	Load at 2.0mm kN	At Proportional Limit		At Maximum		Pm/Pp	Dm/Dp
									Load Pp kN	Tie Displ. Dp mm	Load Pm kN	Tie Displ. Dm mm		
HE-1a	20	20	3/32"	5	N.A.	1.61	0.33	0.57	0.67	2.50	1.64	20.90	2.44	8.36
HE-1b**	16	20	1/4"	5	N.A.	1.29	0.40	0.64	0.82	2.76	1.52	10.70	1.86	3.88
	18	20	1/8"	5	N.A.	1.49	0.32	0.61	0.99	3.54	1.23	8.30	1.24	2.34
	20	20	3/32"	5	N.A.	1.50	0.33	0.60	0.72	2.58	0.87	6.66	1.21	2.58
	21	20	3/32"	5	N.A.	1.87	0.30	0.48	0.46	1.78	0.61	6.68	1.33	3.75
HE-1bA	20	20	3/32"	5	N.A.	1.57	0.30	0.56	0.76	2.88	0.92	10.74	1.22	3.73
HE-1c	16	20	1/4"	5	0.69-0.22	1.09	0.42	0.62	1.52	7.70	1.72	14.30	1.13	1.86
	18	20	1/8"	5	0.2-0.12	1.50	0.34	0.57	1.05	4.82	1.76	23.58	1.68	4.89
	20	20	3/32"	5	.7-.24	2.00	0.28	0.46	0.36	1.32	1.72	17.16	4.78	13.00
	21	20	3/32"	5	1.13-.7	2.04	0.31	0.44	0.31	0.94	1.03	15.82	3.32	16.83
HE-1d**	20	20	3/32"	5	.65-.17	1.49	0.34	0.58	0.70	2.58	0.93	9.90	1.33	3.84
HE-2	16	20	1/4"	5	N.A.	1.48	0.35	0.59	0.74	2.84	1.58	15.82	2.13	5.57
	18	20	1/8"	5	N.A.	2.24	0.22	0.40	0.54	2.92	1.66	22.18	3.06	7.60
	20	20	3/32"	5	N.A.	3.00	0.18	0.31	0.35	2.28	1.65	25.48	4.70	11.18
	21	20	3/32"	5	N.A.	3.84	0.14	0.26	0.27	2.16	1.11	27.04	4.17	12.52
HE-3	20	10	3/32"	5	N.A.	2.13	0.23	0.43	0.62	3.28	1.84	19.38	2.95	5.91
	20	20	3/32"	5	N.A.	3.00	0.18	0.31	0.35	2.28	1.65	25.48	4.70	11.18
	20	30	3/32"	5	N.A.	4.75	0.18	0.27	0.22	1.44	1.70	30.60	7.87	21.25
HE-4**	16	20	1/4"	5	N.A.	1.29	0.41	0.65	0.75	2.40	1.51	10.68	2.02	4.45
	18	20	1/8"	5	N.A.	2.01	0.29	0.48	0.55	2.30	1.34	12.12	2.45	5.27
	20	20	3/32"	5	N.A.	2.98	0.19	0.36	0.46	2.69	0.88	13.16	1.93	4.89
	21	20	3/32"	5	N.A.	3.28	0.19	0.33	0.18	0.88	0.79	13.46	4.49	15.30
HE-4A	20	20	3/32"	5	N.A.	1.98	0.25	0.45	0.70	3.40	0.98	13.44	1.41	3.95
HE-5	18	20	1/8"	5	.76-.49	2.16	0.27	0.43	0.25	0.78	1.88	27.70	7.53	35.51
	20	20	3/32"	5	1.06-.76	2.68	0.24	0.36	0.27	1.04	1.72	25.54	6.46	24.56
	21	20	3/32"	5	1.55-1.31	3.59	0.14	0.27	0.33	2.56	1.04	23.40	3.15	9.14
HE-6**	20	20	3/32"	5	1.06-.88	2.74	0.25	0.40	0.44	2.29	0.92	12.44	2.08	5.42

N - Number of tests conducted

N.A. - Not Applicable

* The first value is the cyclic displacement at 30 cycles and the second value is the cyclic displacement at 1000 cycles.

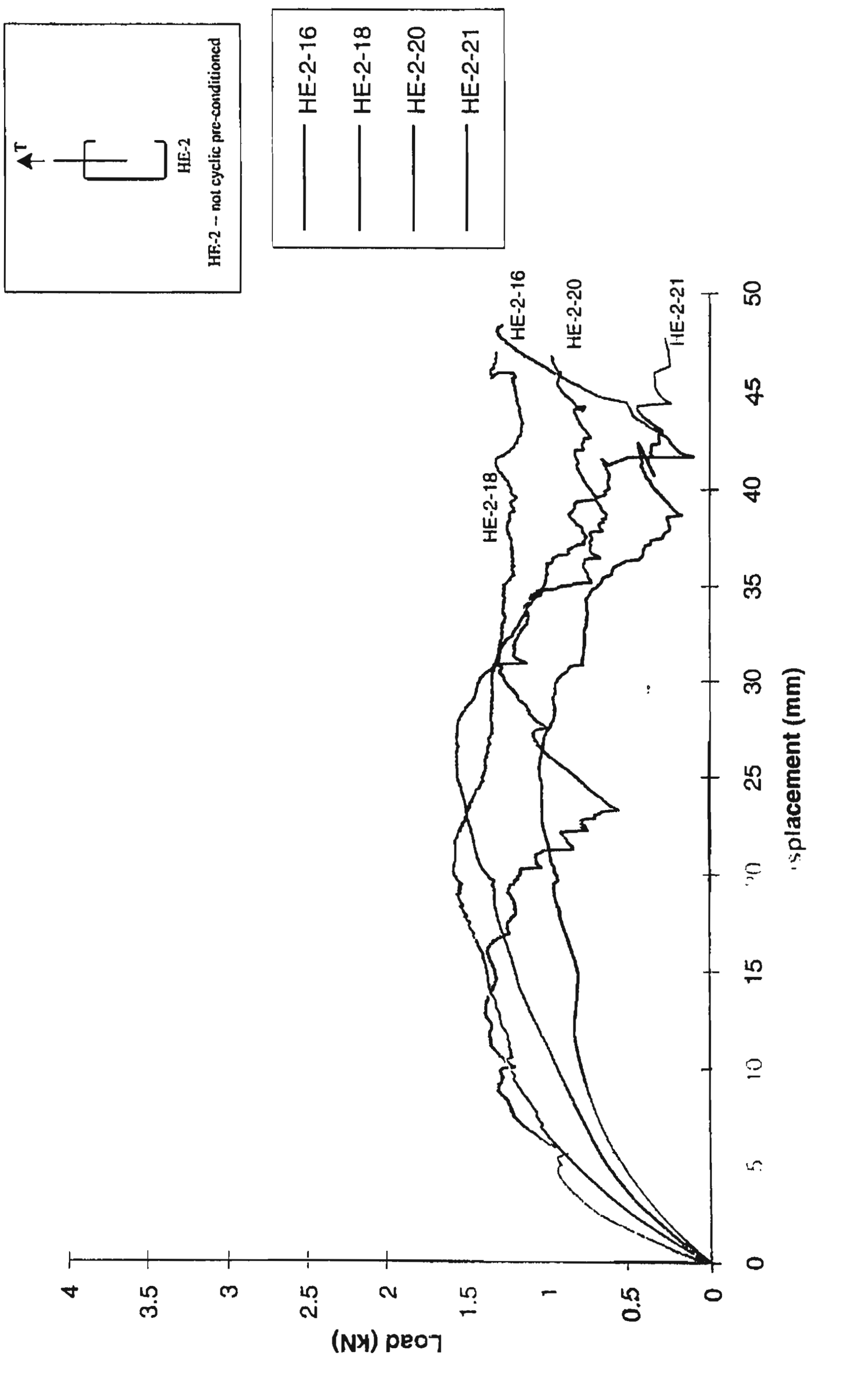
** These maximum values are the values before the load increases due to the tie hitting the interior flange

Table 4-3 : Stiffness Values for Helifix Exterior Repair

Test Series	Gauge	Position	Pilot Hole	N	Stiffness Values based on Displ. at 0.45 kN N/mm	Stiffness Values based on Load at 1 mm N/mm	Stiffness Values based on Load at 2 mm N/mm	Stiffness Values based on Proportional Limit N/mm
HE-1a	20	20	3/32"	5	280	328	286	268
HE-1b	16	20	1/4"	5	349	404	318	296
	18	20	1/8"	5	302	322	304	280
	20	20	3/32"	5	300	328	301	278
	21	20	3/32"	5	241	300	240	258
HE-1bA	20	20	3/32"	5	287	298	281	263
HE-1c	16	20	1/4"	5	413	420	310	197
	18	20	1/8"	5	300	340	285	218
	20	20	3/32"	5	225	280	230	273
	21	20	3/32"	5	221	310	220	330
HE-1d	20	20	3/32"	5	302	342	291	271
HE-2	16	20	1/4"	5	304	346	295	261
	18	20	1/8"	5	201	216	199	186
	20	20	3/32"	5	150	182	157	154
	21	20	3/32"	5	117	142	130	123
HE-3	20	10	3/32"	5	211	228	213	190
	20	20	3/32"	5	150	182	157	154
	20	30	3/32"	5	95	176	134	150
HE-4	16	20	1/4"	5	349	410	324	311
	18	20	1/8"	5	224	288	242	238
	20	20	3/32"	5	151	194	179	170
	21	20	3/32"	5	137	194	165	200
HE-4A	20	20	3/32"	5	227	252	227	205
HE-5	18	20	1/8"	5	208	272	216	321
	20	20	3/32"	5	168	242	181	256
	21	20	3/32"	5	125	136	133	129
HE-6	20	20	3/32"	5	164	248	198	193

N - Number of tests conducted

Figure 4-7 --- Characteristic Curves for Helifix Exterior Repair Test Series 2



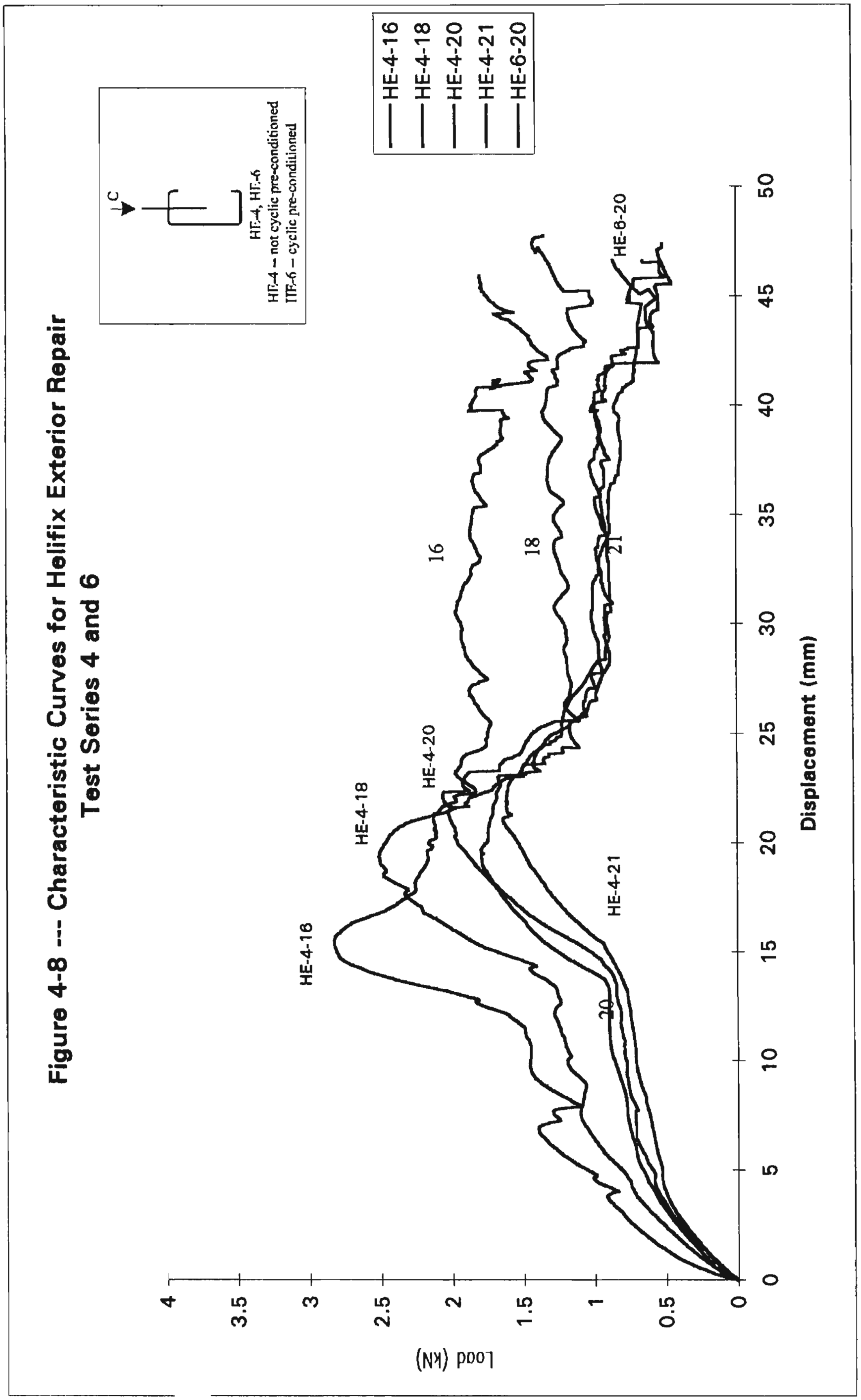


Figure 4-8 --- Characteristic Curves for Helifix Exterior Repair Test Series 4 and 6

4.3.3 Performance Under Cyclic Loading (Series 5 and 6)

The effect of cyclic pre-conditioning on tension performance was, in general, to cause slightly higher maximum loads and initial stiffnesses. Figure 4-9 shows the characteristic curves for tension pullout with and without cyclic pre-conditioning for 18, 20 and 21 gauge studs. As shown, pre-conditioning has little overall effect on performance.

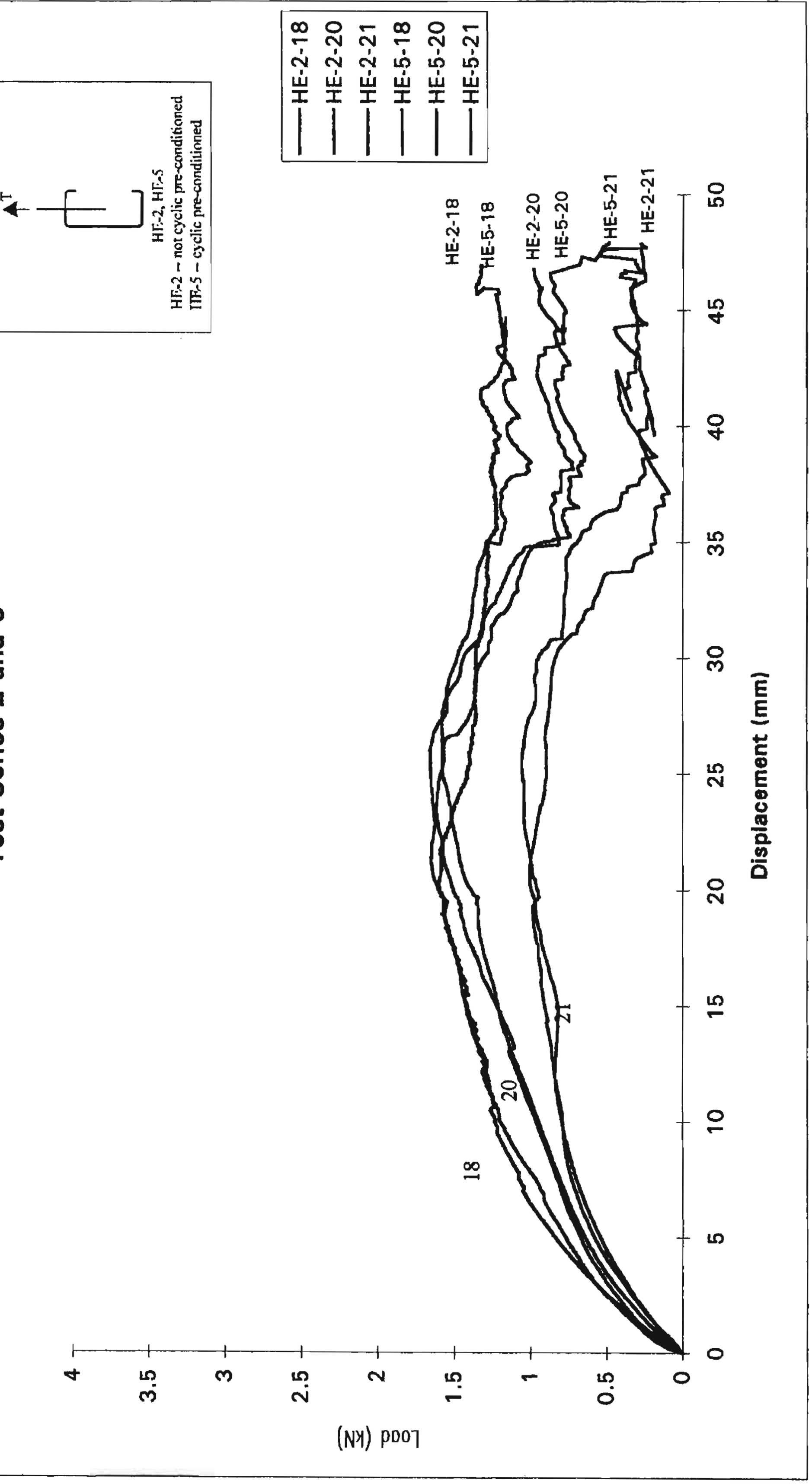
The cyclic displacement, illustrated in Figure 3-11, was recorded early in the cycling regime at approximately 30 cycles and then again at 1000 cycles. Table 4-4 shows the amount of cyclic displacement that resulted with the Helifix Exterior tie for the different stud gauges and test setups. The amount of displacement during cycling decreased significantly with increased number of cycles. As would be expected the cyclic displacements in the beam tests are consistently larger than in the isolation tests. The magnitude of the difference in cyclic displacement between the beam and isolation tests decreases with decreasing steel stud thickness.

Table 4-4 - Displacements During Cyclic Loading for the Helifix Exterior Tie

	16 gauge		18 gauge		20 gauge		21 gauge	
	Isolation Tests mm	Beam Tests mm	Isolation Tests mm	Beam Tests mm	Isolation Tests mm	Beam Tests mm	Isolation Tests mm	Beam Tests mm
at 30 cycles	0.69	N.T.	0.20	0.76	0.70	1.06	1.13	1.55
at 1000 cycles	0.22	N.T.	0.12	0.49	0.24	0.76	0.70	1.31

N.T. - not tested

Figure 4-9 --- Characteristic Curves for Helifix Exterior Repair Test Series 2 and 5



4.3.4 Influence of Stud Gauge

The Helifix Exterior tie connection in 16, 18, and 20 gauge studs had similar maximum strengths in tension, while the 21 gauge had a lower maximum strength. The maximum strength in compression increased with heavier stud gauge. In both tension and compression the initial stiffness increased considerably with increased stud thickness.

4.3.5 Influence of Attachment Location (Series 3)

The influence of attachment location on the flange of the steel stud is illustrated in the characteristic curves in Figure 4-10. As expected, the initial stiffness is very dependent on attachment location. Greater stiffness resulted when the tie was located closer to the stud web. The maximum load for the location closest to the stud web was approximately 10% higher than the other two locations which attained similar strengths.

4.3.6 Comparison of Beam and Isolation Tests

The isolation tests exhibited greater stiffness than the beam tests in both tension and compression. This increase in stiffness was more pronounced with the lighter gauge studs. The test method has a significant influence on the amount of cyclic displacement particularly for the heavier gauge studs. In general, the maximum loads in the isolation tests were 10% higher than in the beam tests for both tension and compression.

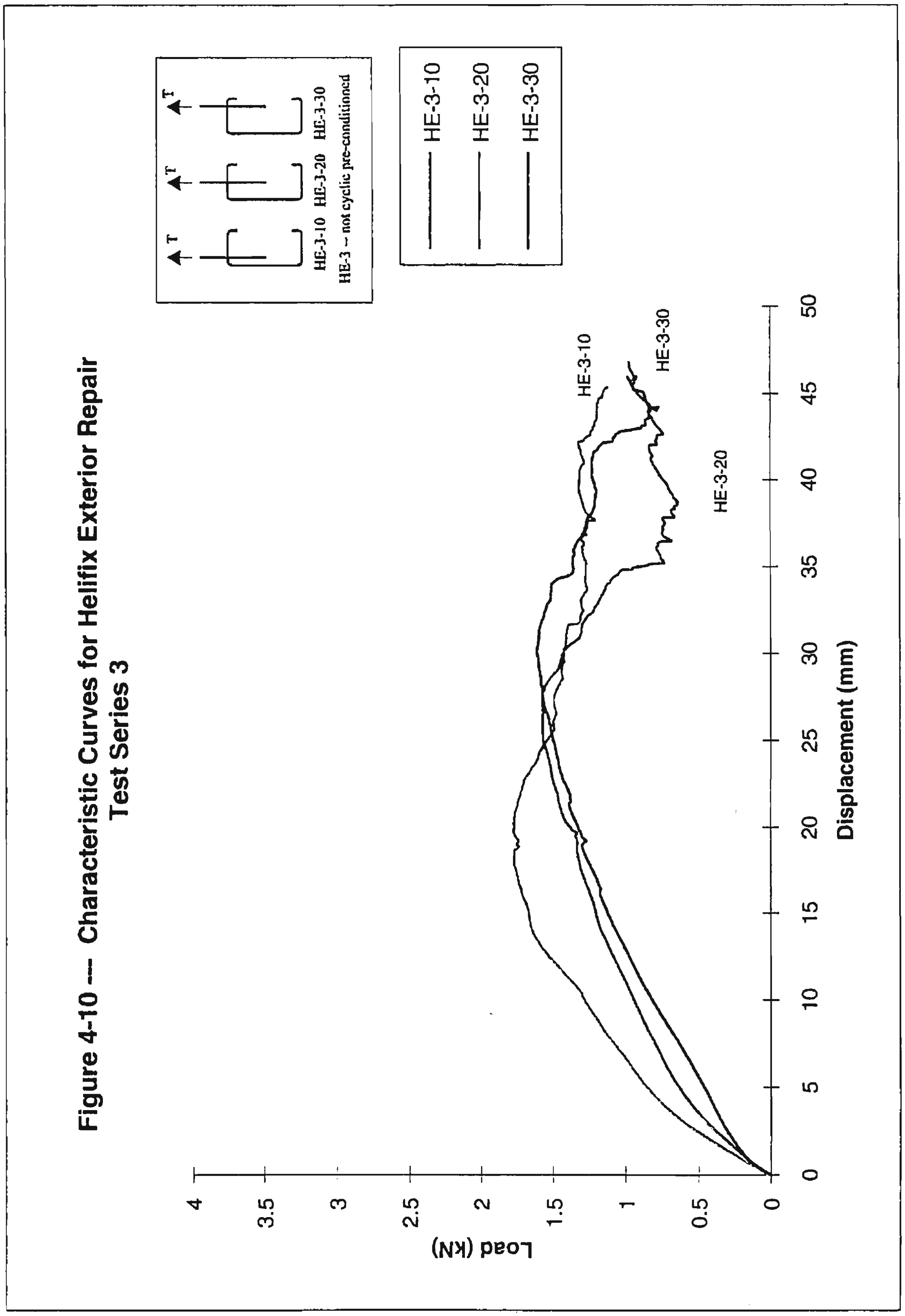


Figure 4-10 — Characteristic Curves for Helifix Exterior Repair Test Series 3

4.4 Dur-O-Wal Exterior Tie

Table 4-5 and 4-6 summarize the results of the tests conducted on the Dur-O-Wal Exterior Tie.

4.4.1 Performance in Tension (Series 2)

Figure 4-11 illustrates the characteristic curves in tension for the Dur-O-Wal Exterior tie. The pullout performance of the Dur-O-Wal Exterior tie in tension may be characterized as initially linear but, after 1 to 2 mm of displacement, becoming increasingly non-linear. Unloading, after the maximum load has been attained, is very sudden with no reliable ductility past the maximum load level. The initial stiffness and the maximum load increase significantly with increased steel stud thickness.

4.4.2 Performance in Compression (Series 4)

The characteristic curves in compression for the Dur-O-Wal Exterior tie are shown in Figure 4-12. There is an initial range of linear response followed by significant nonlinear deformation with the load remaining relatively constant. The heavier gauge studs permitted significantly higher maximum loads and showed greater initial stiffness. The 20 and 21 gauge studs do show relatively similar stiffnesses up to 2 mm of deformation after which the 20 gauge stud is more stiff. The pullout response of the Dur-O-Wal Exterior tie was much stiffer in tension than in compression. In compression the deformation includes the deflection and rotation of the steel stud section. Tie deformation was negligible as the tie bears directly on the flange of the steel stud. The greater stiffness in tension is due, in part, to the increased stiffness that the steel stud flange exhibits as was also shown with the Rigid Datum connection.

Table 4-5 : Average Values for the Dur-O-Wal Exterior Repair

Test Series	Gauge	Position	Pilot Hole	N	Cyclic Displ.* mm	Displ. at 0.45kN mm	Load at 1.0 mm kN	Load at 2.0mm kN	At Proportional Limit Load Pp kN	At Proportional Limit Tie Displ. Dp mm	At Maximum Load Pm kN	At Maximum Tie Displ. Dm mm	Pm/Pp	Dm/Dp
DE-1a	20	20	5/32"	5	N.A.	0.31	1.27	1.97	1.43	1.24	2.57	3.44	1.80	2.77
DE-1b	20	20	5/32"	3	N.A.	0.44	0.93	1.66	5.00	6.83	5.00	6.83	1.00	1.00
DE-1c	16	20	3/16"	5	0.03	0.13	2.48	3.33	2.21	0.80	4.33	3.65	1.96	4.56
	18	20	5/32"	5	0.08-0.07	0.21	1.53	2.26	1.34	0.78	3.22	3.74	2.40	4.79
	20	20	5/32"	5	0.16-0.14	0.39	1.12	1.73	1.29	1.22	2.43	3.33	1.88	2.73
	21	20	5/32"	5	0.26-0.24	0.88	0.5	0.82	0.42	0.72	1.75	4.84	4.17	6.72
DE-2	16	20	3/16"	5	N.A.	0.28	1.53	2.31	1.80	1.26	4.05	6.72	2.26	5.31
	18	20	5/32"	5	N.A.	0.53	0.85	1.34	1.00	1.23	2.67	8.43	2.66	6.88
	20	20	5/32"	5	N.A.	0.89	0.50	0.84	0.65	1.40	2.17	10.36	3.32	7.39
	21	20	5/32	5	N.A.	1.21	0.41	0.61	0.39	0.96	1.37	8.96	3.49	9.34
DE-3	20	10	3/16"	5	N.A.	0.21	1.47	2.12	1.43	0.95	2.36	3.17	1.65	3.33
	20	20	5/32"	5	N.A.	0.89	0.50	0.84	0.65	1.40	2.17	10.36	3.32	7.39
	20	30	5/32"	5	N.A.	1.79	0.28	0.49	0.47	1.83	2.02	14.12	4.32	7.70
DE-4	18	20	5/32"	5	N.A.	0.54	0.72	1.21	0.95	1.49	3.54	19.08	3.74	12.77
	20	20	5/32"	5	N.A.	1.24	0.39	0.66	0.52	1.52	2.42	18.40	4.61	12.09
	21	20	5/32	5	N.A.	1.30	0.39	0.60	0.30	0.66	1.42	15.06	4.73	22.96
DE-5	18	20	5/32"	5	0.27	0.54	0.82	1.27	0.98	1.29	2.68	9.33	2.73	7.22
	20	20	5/32"	5	0.57	0.99	0.46	0.78	0.66	1.55	2.06	9.62	3.12	6.21
	21	20	5/32"	5	0.49	1.23	0.39	0.62	0.39	0.96	1.34	7.97	3.48	8.29
DE-6	20	20	5/32"	5	0.59	1.15	0.41	0.70	0.48	1.25	2.61	19.48	5.40	15.58

N - Number of tests conducted
N.A. - Not Applicable
* The first value is cyclic displacement at 30 cycles and the second value is the cyclic displacement at 1000 cycles.

Table 4-6 : Stiffness Values for the Dur-O-Wal Exterior Repair

Test Series	Gauge	Position	Pilot Hole	N	Stiffness Values based on Displ. at 0.45 kN N/mm	Stiffness Values based on Load at 1 mm N/mm	Stiffness Values based on Load at 2 mm N/mm	Stiffness Values based on Proportional Limit N/mm
DE-1a	20	20	5/32"	5	1452	1268	983	1152
DE-1b	20	20	5/32"	3	1023	930	832	732
DE-1c	16	20	3/16"	5	3462	2480	1665	2763
	18	20	5/32"	5	2143	1530	1130	1718
	20	20	5/32"	5	1154	1120	865	1057
	21	20	5/32"	5	511	500	410	583
DE-2	16	20	3/16"	5	1607	1532	1153	1421
	18	20	5/32"	5	849	850	672	819
	20	20	5/32"	5	506	502	422	466
	21	20	5/32	5	372	408	304	408
DE-3	20	10	3/16"	5	2143	1468	1058	1495
	20	20	5/32"	5	506	502	422	466
	20	30	5/32"	5	251	280	247	255
DE-4	18	20	5/32"	5	833	722	603	635
	20	20	5/32"	5	363	390	332	344
	21	20	5/32	5	346	394	302	457
DE-5	18	20	5/32"	5	833	816	635	760
	20	20	5/32"	5	455	462	390	425
	21	20	5/32"	5	366	394	311	401
DE-6	20	20	5/32"	5	391	412	349	387

N - Number of tests conducted

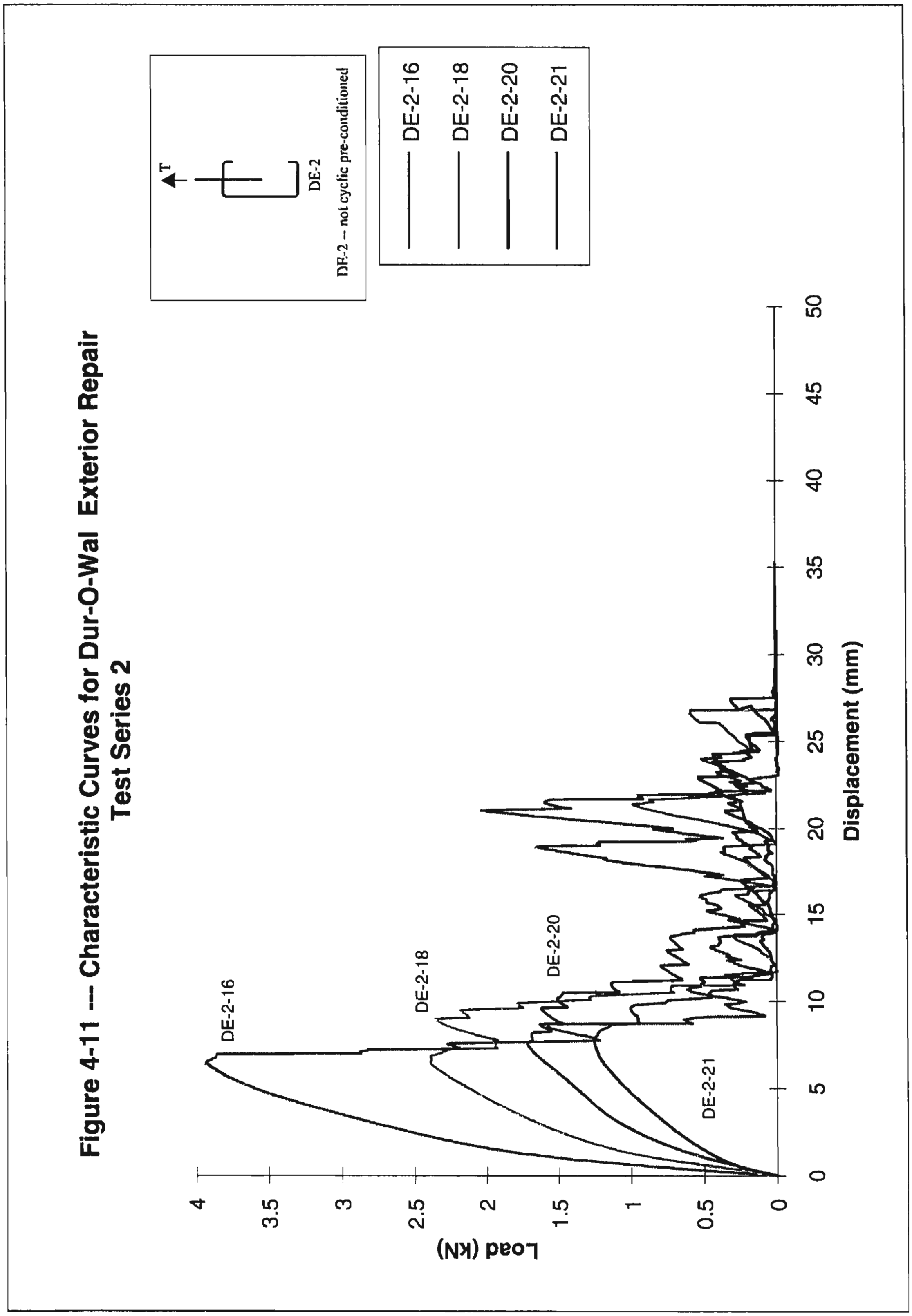


Figure 4-11 --- Characteristic Curves for Dur-O-Wal Exterior Repair Test Series 2

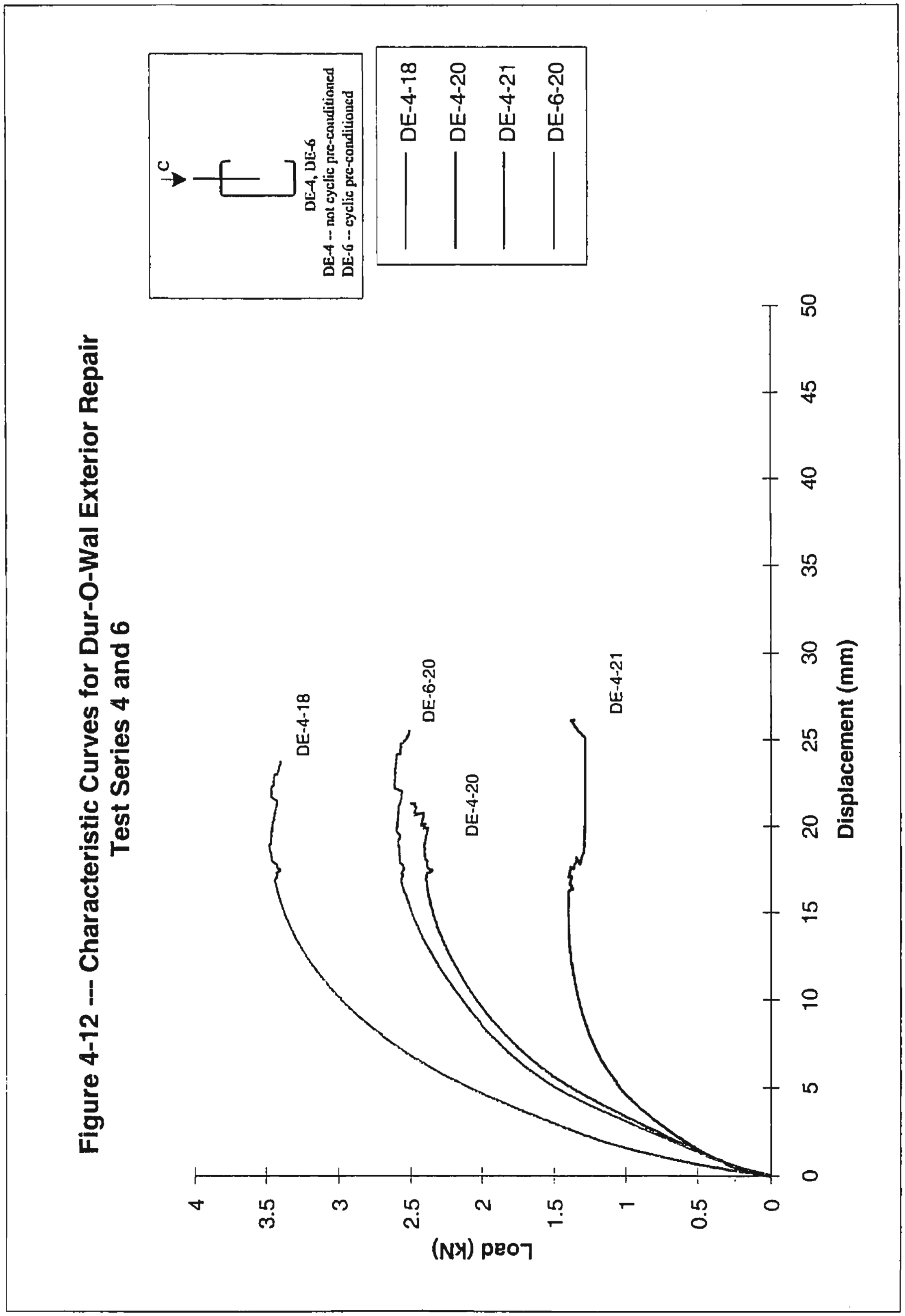


Figure 4-12 --- Characteristic Curves for Dur-O-Wal Exterior Repair Test Series 4 and 6

4.4.3 Performance Under Cyclic Loading (Series 5 and 6)

Figure 4-13 shows the characteristic curves for tension pullout with and without cyclic pre-conditioning for 18, 20 and 21 gauge studs. As shown, cyclic pre-conditioning did not alter the performance of the Dur-O-Wal Exterior tie in any noticeable way.

Curve DE-6-20 in Figure 4-12 shows the characteristic curve for the Dur-O-Wal Exterior Tie tested in compression after cyclic loading. Only the 20 gauge stud connection was tested in compression after cyclic pre-conditioning. The results show that little difference in performance results due to pre-conditioning.

The cyclic displacement, illustrated in Figure 3-11, was recorded early in the cycling regime -- at approximately 30 cycles and then again at 1000 cycles. Table 4-7 shows the amount of cyclic displacement that occurred with the Dur-O-Wal Exterior tie for the different stud gauges and test setups. The amount of displacement during cycling remained constant with increased number of cycles. As would be expected the cyclic displacements in the beam tests are consistently larger than in the isolation tests. The lower amount of cyclic displacement in the beam tests for the 21 gauge stud relative to the 20 gauge is possible considering that the flange width of the 21 gauge stud is only 31 mm compared to 41 mm for the 20 gauge stud.

Table 4-7 - Displacements During Cyclic Loading for the Dur-O-Wal Exterior Tie

	16 gauge		18 gauge		20 gauge		21 gauge	
	Isolation Tests mm	Beam Tests mm	Isolation Tests mm	Beam Tests mm	Isolation Tests mm	Beam Tests mm	Isolation Tests mm	Beam Tests mm
at 30 cycles	0.03	N.T.	0.08	0.27	0.16	0.57	0.26	0.49
at 1000 cycles	0.03	N.T.	0.07	0.27	0.14	0.57	0.24	0.49

N.T. - not tested

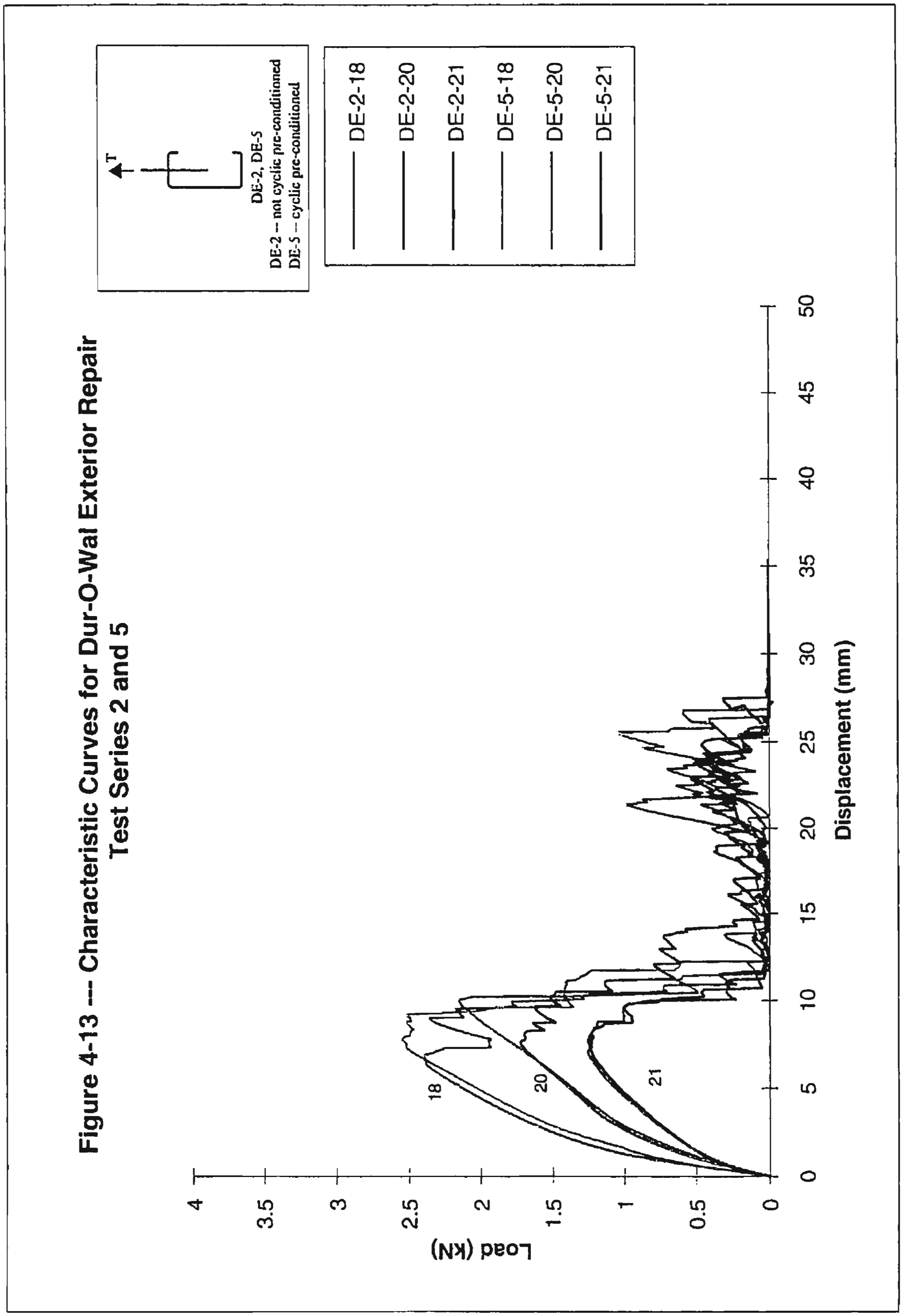


Figure 4-13 --- Characteristic Curves for Dur-O-Wal Exterior Repair Test Series 2 and 5

4.4.4 Influence of Stud Gauge

The Dur-O-Wal Exterior tie performed very differently in tension and compression. Increases in stud thickness resulted in higher maximum loads, higher initial stiffnesses and decreased cyclic displacement. The Dur-O-Wal exterior connection fails in tension with the lagbolt tearing out of the steel stud flange. In compression, failure is due to excessive lateral deformation of the steel stud cross-section. This pattern of failure in tension and compression did not change with the different stud gauges.

4.4.5 Influence of Attachment Location (Series 3)

The influence of attachment location with regard to the flange a 20 gauge steel stud is illustrated in the characteristic curves in Figure 4-14. As expected, the initial stiffness is very dependent on attachment location with greater stiffness when the tie is located closer to the stud web. This increased stiffness results in the loads at 1 and 2 mm being very different e.g. for positions 10, 20 and 30 mm away from the web the load at 2 mm was 2.12, 0.84 and 0.49 respectively. The average maximum load increased by approximately 10 % for every 10 mm closer to the stud web that the tie was installed.

4.4.6 Comparison of Beam and Isolation Tests

The isolation tests exhibited greater stiffness than the beam tests in both tension and compression. This increase in stiffness was least for the 21 gauge studs. The influence of the test method on the amount of cyclic displacement was significant for all gauges of studs. The maximum loads in the isolation tests were 10-20 % higher than in the beam tests when tested in tension.

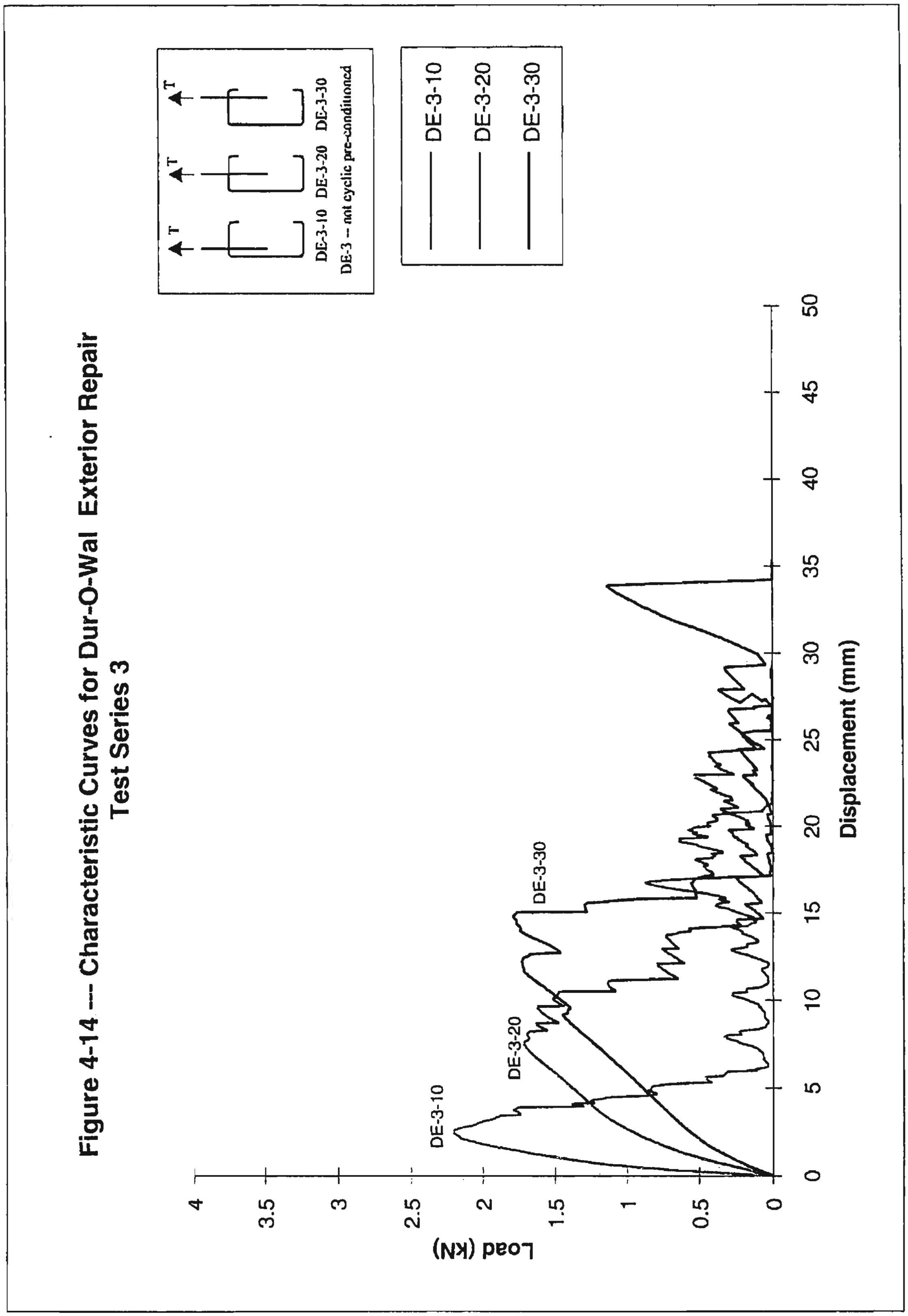


Figure 4-14 --- Characteristic Curves for Dur-O-Wal Exterior Repair Test Series 3

4.5 Helifix Interior Tie

Table 4-8 and 4-9 summarize the results of the tests conducted on the Helifix Interior tie. It is important to note that the pilot hole diameters are the same for the different gauges of stud when using the Helifix tie from the interior.

4.5.1 Performance in Tension (Series 2)

The performance of the Helifix Interior tie in tension is characterized by an initially linear pullout response followed by significant non-linear ductile pullout response. Figure 4-15, the characteristic curves for the four stud gauges, illustrates this ductility. The initial stiffness increases with increased steel stud thickness. The maximum load occurred at approximately 10 mm. of displacement. The maximum load occured at 10mm of displacement because the Helifix Interior tie only penetrates the interior flange of the stud by 10mm. Thus when loaded in tension the tie pulled out of the interior flange of the steel stud and the connection to the steel stud was limited to one flange. Ductile and stable response was observed even after the tie pulled out of one flange. Ultimately pullout occured with more damage to the tie with the thicker studs and more damage to the flange with the thinner studs.

4.5.2 Performance in Compression (Series 4)

The performance of the Helifix Interior tie in compression is illustrated in Figure 4-16. There is an initial range of linear response followed by deformation with the load remaining relatively constant. For 20 and 21 gauge studs the load decreases significantly after approximately 30 mm of displacement whereas the ties into the 16 and 18 gauge studs still continued to carry near maximum loads up to displacements of 45 mm., at which point the test was stopped. The initial stiffness of the tie connection in compression was approximately the same as that in tension.

The Helifix Interior tie connection attained maximum loads that were 17, 60, 50, and 25 percent higher in compression than in tension for the 16, 18, 20 and 21 gauge connections respectively. One reason is because the compression loading is opposite to the direction of installation of the tie and pre-drilling from the inside deforms the metal outwards. Note that the initial stiffness values are the same for both tension and compression.

Table 4-8 : Average Values for the Helifix Interior Repair

Test Series	Gauge	Position	Pilot Hole	N	Cyclic Displ. mm	Displ. at 0.45 kN mm	Load at 1.0 mm kN	Load at 2.0mm kN	At Proportional Limit: Load Pp kN	At Proportional Limit: Tie Displ. Dp mm	At Maximum: Load Pm kN	At Maximum: Tie Displ. Dm mm	Pm/Pp	Dm/Dp
HI-1A	20	20	1/4"	5	N.A,	1.66	0.37	0.51	0.41	1.10	1.21	10.76	2.93	9.78
HI-1B	16	20	1/4"	3	N.A,	0.77	0.44	0.66	0.49	0.97	2.05	20.03	4.16	20.72
	18	20	1/4"	5	N.A,	1.45	0.34	0.60	0.91	3.56	2.22	18.26	2.45	5.13
	20	20	1/4"	5	N.A,	1.63	0.34	0.55	0.74	3.06	1.75	16.18	2.35	5.29
	21	20	1/4"	5	N.A,	2.18	0.28	0.46	0.46	1.96	1.08	18.16	2.34	9.27
HI-1C	16	20	1/4"	5	0.02	1.07	0.53	0.84	0.93	2.24	1.86	11.30	2.00	5.04
	18	20	1/4"	5	0.11-0.04	1.45	0.36	0.59	0.78	2.80	1.77	12.98	2.27	4.64
	20	20	1/4"	5	0.47-0.13	1.76	0.29	0.50	0.63	2.50	1.23	9.74	1.96	3.90
	21		1/4"	5	0.35-0.21	2.79	0.28	0.39	0.31	1.30	0.85	9.30	2.78	7.15
HI-1D	20	20	1/4"	5	.41-.15	1.34	0.40	0.59	0.38	0.46	1.71	18.88	4.54	41.04
HI-2	16	20	1/4"	5	N.A,	0.58	0.66	0.92	0.77	1.04	1.78	10.78	2.33	10.37
	18	20	1/4"	5	N.A,	1.62	0.32	0.54	0.83	3.42	1.39	10.68	1.68	3.12
	20	20	1/4"	5	N.A,	2.1	0.21	0.42	0.65	3.24	1.10	10.04	1.69	3.10
	21	20	1/4"	5	N.A,	2.63	0.20	0.36	0.53	3.24	0.88	10.10	1.65	3.12
HI-3	20	10	1/4"	5	N.A,	1.51	0.33	0.53	0.63	2.52	1.22	9.94	1.92	3.94
	20	20	1/4"	5	N.A,	2.1	0.21	0.42	0.65	3.24	1.10	10.04	1.69	3.10
	20	30	1/4"	5	N.A,	2.33	0.22	0.39	0.70	4.00	1.13	10.86	1.61	2.72
HI-4	16	20	1/4"	4	N.A,	0.37	0.72	0.88	0.63	0.63	2.08	15.03	3.29	24.04
	18	20	1/4"	5	N.A,	1.58	0.33	0.54	0.69	2.74	2.22	20.72	3.24	7.56
	20	20	1/4"	5	N.A,	2.41	0.22	0.42	0.55	2.80	1.65	20.26	3.03	7.24
	21	20	1/4"	4	N.A,	2.73	0.22	0.37	0.48	2.95	1.10	19.93	2.28	6.75
HI-5	18	20	1/4"	5	.41-.28	1.77	0.27	0.52	0.67	2.84	1.44	12.96	2.16	4.56
	20	20	1/4"	5	.82-.40	2.09	0.24	0.43	0.61	3.02	1.16	11.84	1.89	3.92
	21	20	1/4"	5	1.01-.55	2.85	0.20	0.34	0.53	3.36	0.92	12.26	1.75	3.65
HI-6	20	20	1/4"	4	.53-.36	2.13	0.30	0.45	0.26	0.60	1.65	17.53	6.30	29.21

N - Number of tests conducted

N.A. - Not Applicable

* The first value is the cyclic displacement at 30 cycles and the second value is the cyclic displacement at 1000 cycles.

Table 4-9 : Stiffness Values for the Helifix Interior Repair

Test Series	Gauge	Position	Pilot Hole	N	Stiffness Values based on Displ. at 0.45 kN N/mm	Stiffness Values based on Load at 1 mm N/mm	Stiffness Values based on Load at 2 mm N/mm	Stiffness Values based on Proportional Limit N/mm
HI-1A	20	20	1/4"	5	271	374	253	376
HI-1B	16	20	1/4"	3	584	443	330	510
	18	20	1/4"	5	310	344	298	254
	20	20	1/4"	5	276	336	274	243
	21	20	1/4"	5	206	280	228	235
HI-1C	16	20	1/4"	5	421	526	421	415
	18	20	1/4"	5	310	358	294	278
	20	20	1/4"	5	256	292	250	250
	21		1/4"	5	161	278	194	235
HI-1D	20	20	1/4"	5	336	402	296	817
HI-2	16	20	1/4"	5	776	664	459	737
	18	20	1/4"	5	278	316	270	243
	20	20	1/4"	5	214	208	212	201
	21	20	1/4"	5	171	204	182	164
HI-3	20	10	1/4"	5	298	330	267	251
	20	20	1/4"	5	214	208	212	201
	20	30	1/4"	5	193	218	195	175
HI-4	16	20	1/4"	4	1216	718	439	1012
	18	20	1/4"	5	285	328	270	250
	20	20	1/4"	5	187	218	211	195
	21	20	1/4"	4	165	215	183	164
HI-5	18	20	1/4"	5	254	268	258	235
	20	20	1/4"	5	215	242	216	203
	21	20	1/4"	5	158	196	169	157
HI-6	20	20	1/4"	4	211	303	223	438

N - Number of tests conducted

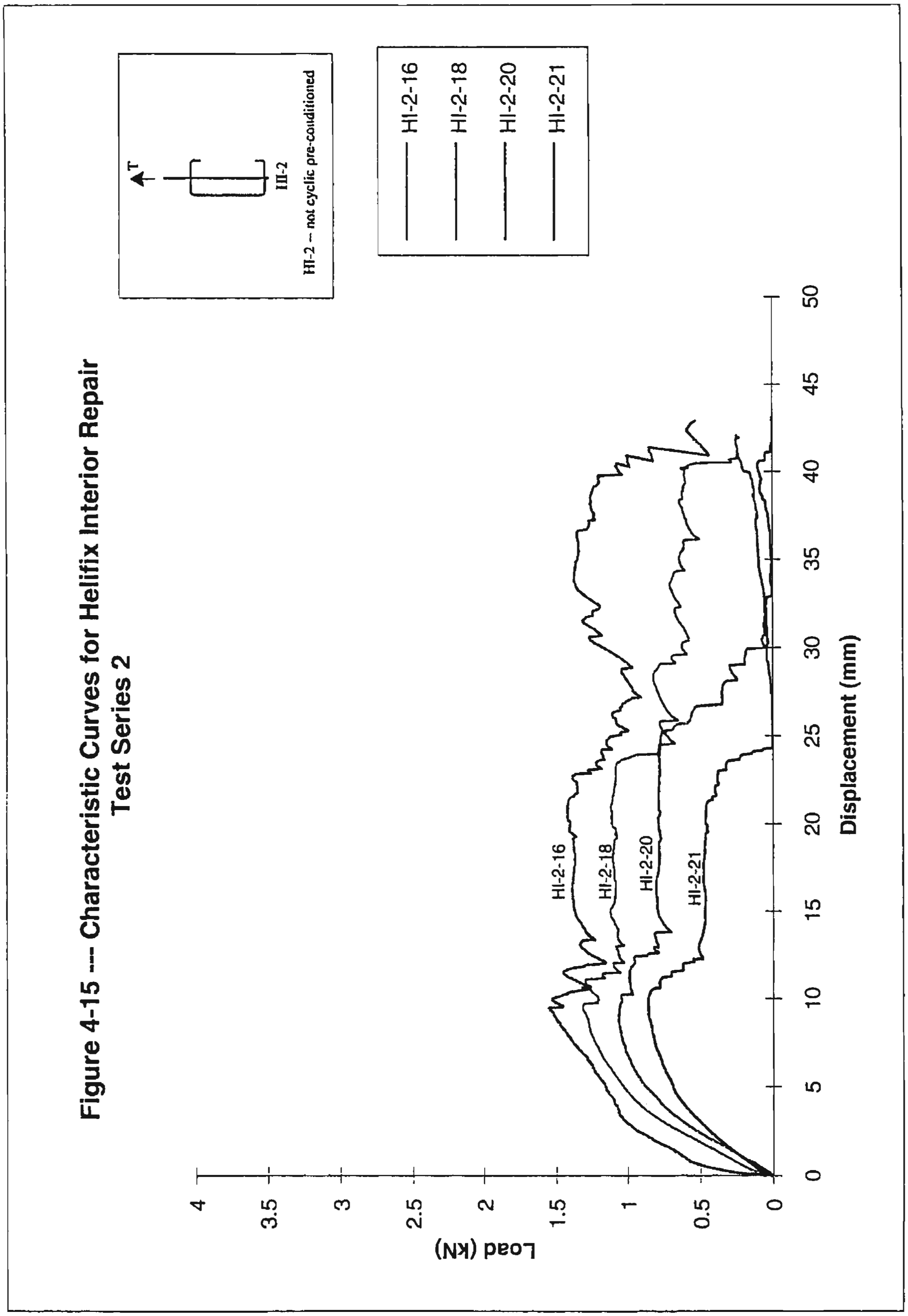


Figure 4-15 --- Characteristic Curves for Helifix Interior Repair Test Series 2

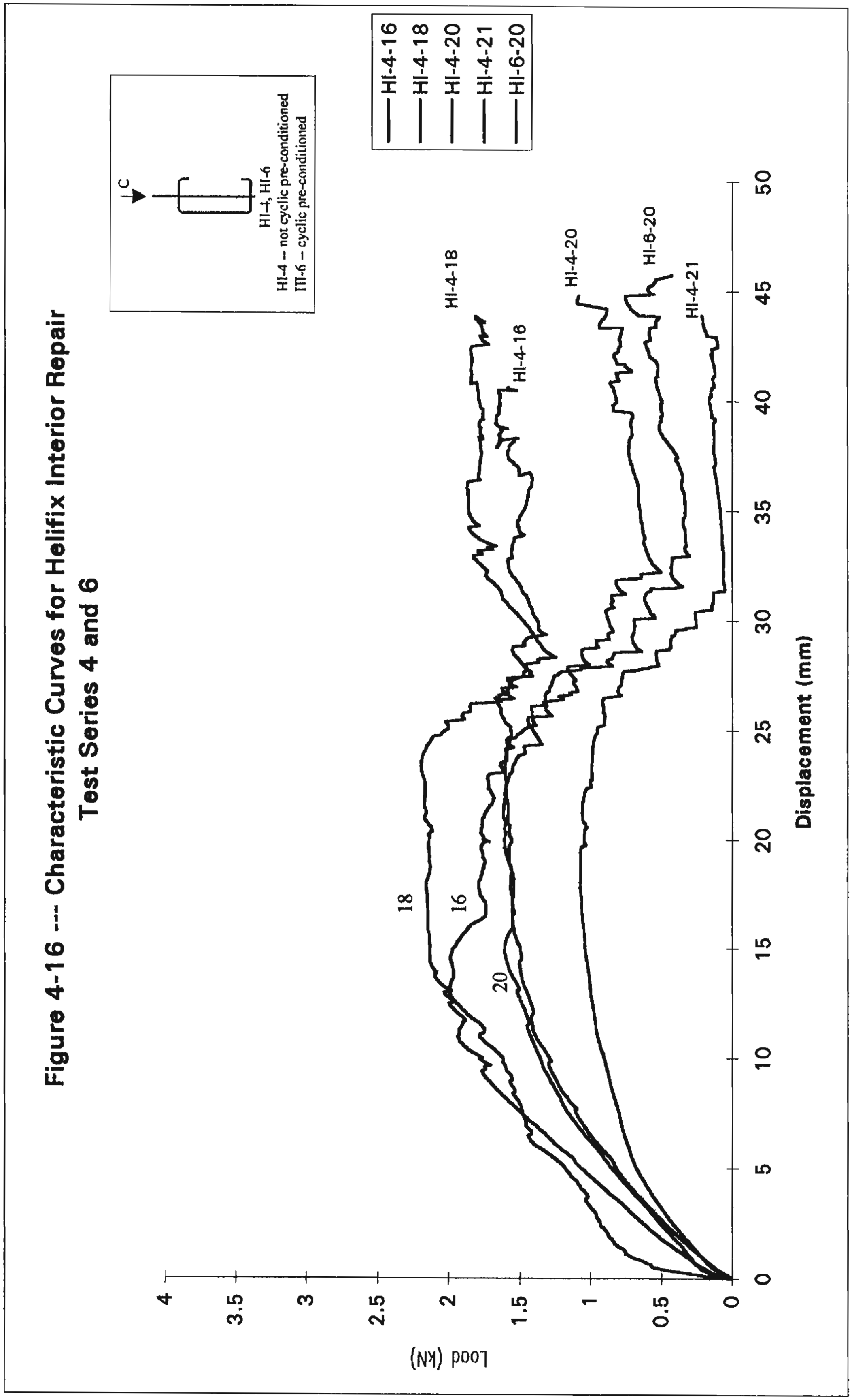


Figure 4-16 --- Characteristic Curves for Helifix Interior Repair Test Series 4 and 6

4.5.3 Performance Under Cyclic Loading (Series 5 and 6)

Cyclic pre-conditioning did not affect the tension or compression maximum loads nor the initial stiffness of the Helifix Interior connection. Figure 4-16 shows the characteristic curves for tension pullout with and without cyclic pre-conditioning for 18, 20 and 21 gauge studs. As shown little overall effect on performance results due to preconditioning.

Only wirh the 20 gauge studs were ties tested in compression after cyclic loading. The results show that little difference results due to pre-conditioning. Figure 4-15 shows the cyclic characteristic curve HI-6-20 superimposed on the non-preconditioned curves.

The cyclic displacement, illustrated in Figure 3-11, was recorded early in the cycling regime, at approximately 30 cycles and then again at 1000 cycles. Table 4-7 shows the amount of cyclic displacement that occured for the Helifix Interior tie for the different stud gauges and test setups. The extent of displacement during cycling decreases significantly with increased number of cycles. As would be expected, the cyclic displacements in the beam tests were consistently larger than in the isolation tests.

Table 4-10 - Displacements During Cyclic Loading for the Helifix Interior Tie

	16 gauge		18 gauge		20 gauge		21 gauge	
	Isolation Tests mm	Beam Tests mm	Isolation Tests mm	Beam Tests mm	Isolation Tests mm	Beam Tests mm	Isolation Tests mm	Beam Tests mm
at 30 cycles	0.02	N.T.	0.11	0.41	0.47	0.82	0.35	1.01
at 1000 cycles	0.02	N.T.	0.04	0.28	0.13	0.40	0.21	0.55

N.T. - not tested

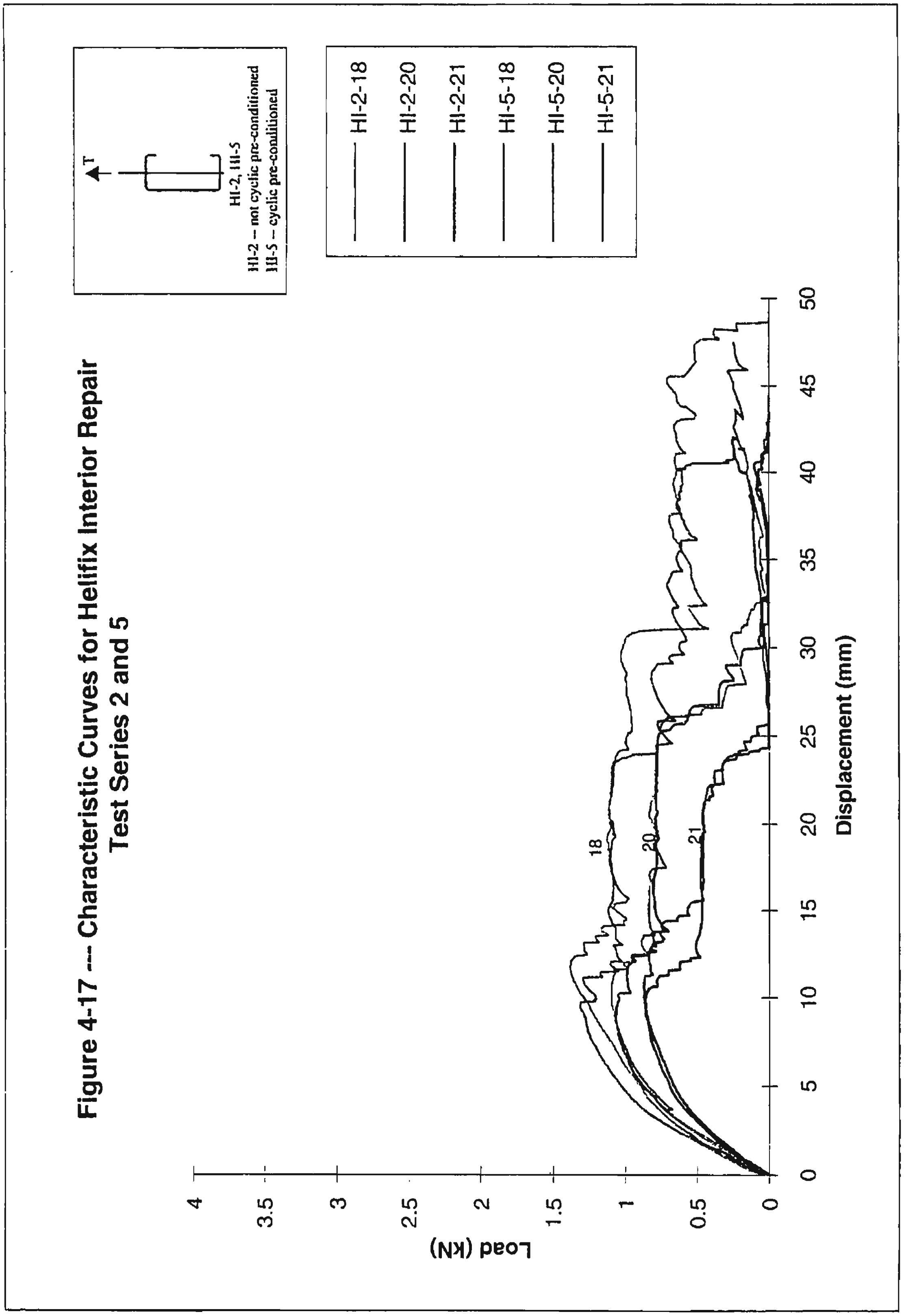


Figure 4-17 --- Characteristic Curves for Helifix Interior Repair Test Series 2 and 5

4.5.4 Influence of Stud Gauge

The performance of the Helifix Interior tie was improved with increased stud thickness in regards to initial stiffness, maximum strength and cyclic displacement.

4.5.5 Influence of Attachment Location (Series 3)

As shown in the characteristic curves for different attachment locations in Figure 4-18, there was very little change in performance for different attachment locations on the flange of the steel stud flange.

4.5.6 Comparison of Beam and Isolation Tests

The values for maximum strength and the linear initial stiffness for the isolation tests were very similar to the values from the beam tests.

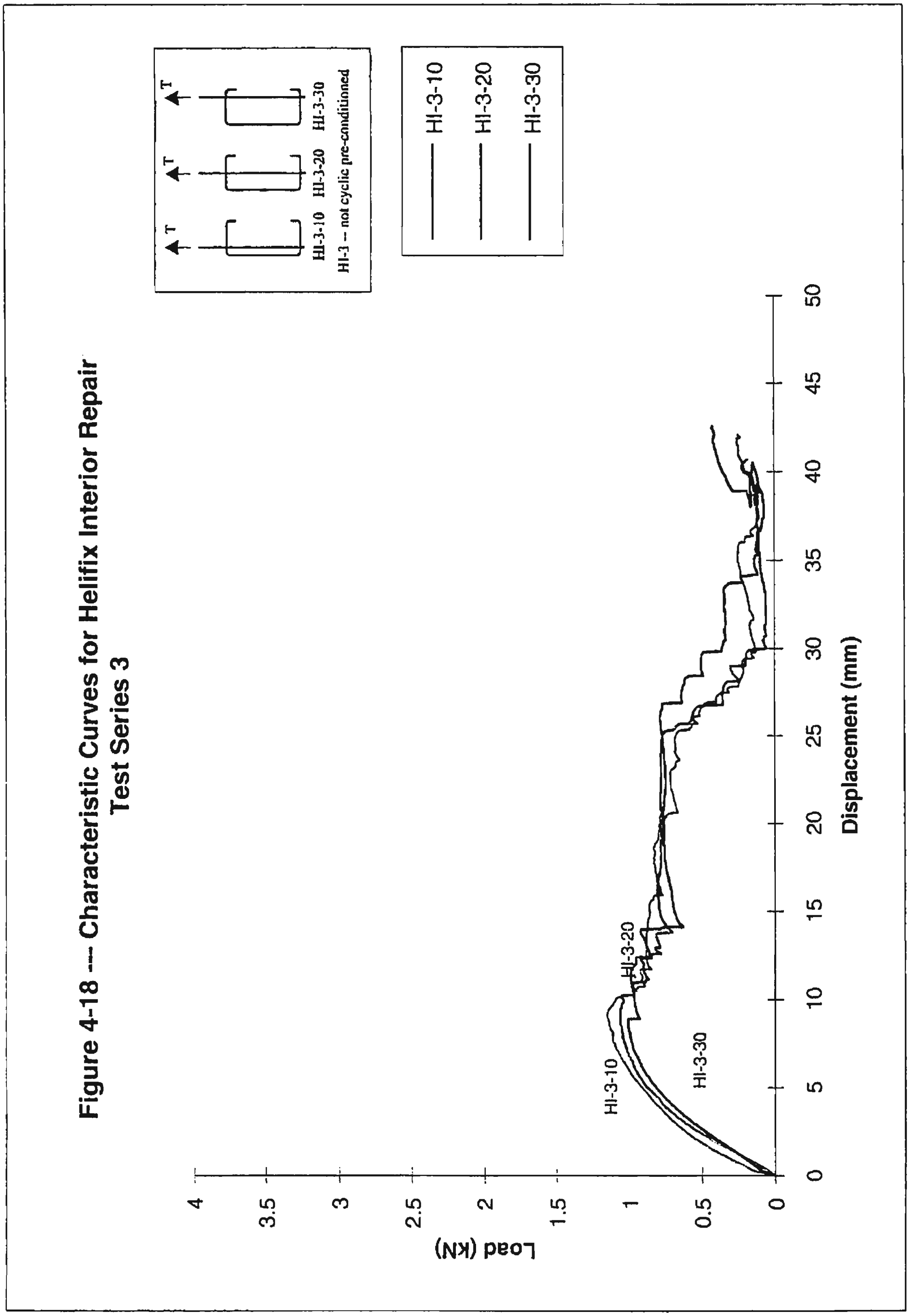


Figure 4-18 --- Characteristic Curves for Helifix Interior Repair Test Series 3

4.6 Dur-O-Wal Interior Tie

Table 4-11 and 4-12 summarize the results of the tests conducted on the Dur-O-Wal Interior tie.

4.6.1 Performance in Tension (Series 2)

In Figure 4-19 the characteristic curves for the 18, 20 and 21 gauge tests are shown. The performance of the Dur-O-Wal Interior tie in tension is characterized by an initially linear range of response followed by non-linear response up to a peak load. With thicker studs there is then a sharp decrease in load. This tie initially carries the load solely by the connection of the epoxy to the steel stud. After a small amount of displacement the nut on the epoxied rod bears directly on the steel stud and carries part of the load. The maximum strength of the tie was not necessarily determined as each test was stopped at 25 mm of displacement. With the heavier gauge studs the epoxy near the outer flange spalled resulting in the load spikes shown in Figure 4-19.

4.6.2 Performance in Compression (Series 4)

The performance of the Dur-O-Wal Interior tie in compression is very different from that in tension. The representative pullout curve for the four gauges is illustrated in Figure 4-20. Initially response is linear followed by highly variable behaviour. Intermittent brittle spalling of the epoxy contributes to the unsteady and unpredictable nature of the curve. The maximum load in compression was nearly the same for all gauges tested as failure occurred by spalling of the epoxy. The tie was slightly stiffer in tension than in compression for the gauges tested (18, 20 and 21).

Table 4-11 : Average Values for the Dur O Wal Interior Repair

									At Proportional Limit		At Maximum			
Test Series	Gauge	Position	Pilot Hole	N	Cyclic Displ. mm	Displ. at 0.45 kN mm	Load at 1.0 mm kN	Load at 2.0mm kN	Load Pp kN	Tie Displ. Dp mm	Load Pm kN	Tie Displ. Dm mm	Pm/Pp	Dm/Dp
DI-1b	16	20	3/8"	5	N.A.	0.11	2.31	2.09	1.82	0.52	2.63	23.84	1.45	45.85
	18	20	3/8"	5	N.A.	0.20	1.37	1.80	1.47	0.80	3.31	13.86	2.26	17.33
	20	20	3/8"	5	N.A.	0.27	0.93	1.13	0.98	0.72	4.57	33.82	4.68	46.97
	21	20	3/8"	5	N.A.	0.28	0.85	1.16	0.74	0.62	2.98	32.54	4.04	52.48
DI-1d	20	20	3/8"	5	0.06	0.31	1.11	1.12	1.12	0.94	4.82	29.26	4.31	31.13
DI-2	18	20	3/8"	5	N.A.	0.30	1.46	2.43	2.03	1.52	5.16	11.95	2.53	7.86
	20	20	3/8"	5	N.A.	0.35	1.14	1.90	1.43	1.33	4.45	9.96	3.11	7.51
	21	20	3/8"	5	N.A.	0.51	0.79	1.09	0.76	0.96	2.53	15.36	3.31	16.00
DI-4	16	20	3/8"	4	N.A.	0.17	1.23	1.14	1.21	0.58	2.63	10.80	2.17	18.78
	18	20	3/8"	5	N.A.	0.27	1.16	1.39	1.42	1.08	2.20	9.80	1.55	9.07
	20	20	3/8"	5	N.A.	0.62	0.67	0.88	0.79	1.10	2.33	12.74	2.94	11.58
	21	20	3/8"	5	N.A.	0.56	0.66	0.78	0.60	0.84	2.19	20.90	3.62	24.88
DI-6	18	20	3/8'"	5	0.11	0.48	0.59	1.03	0.54	0.60	1.87	5.78	3.44	9.63
	20	20	3/8'"	5	0.29	1.18	0.38	0.65	0.49	0.86	1.83	9.88	3.77	11.49
	21	20	3/8'"	5	0.23	0.61	0.56	0.53	0.52	0.76	1.57	16.02	3.00	21.08

N - Number of tests conducted
N.A. - Not Applicable

Table 4-12 : Stiffness Values for the Dur O Wal Interior Repair

Test Series	Gauge	Position	Pilot Hole	N	Stiffness Values based on Displ. at 0.45 kN N/mm	Stiffness Values based on Load at 1 mm N/mm	Stiffness Values based on Load at 2 mm N/mm	Stiffness Values based on Proportional Limit N/mm
DI-1b	16	20	3/8"	5	4091	2308	1046	3492
	18	20	3/8"	5	2250	1366	899	1833
	20	20	3/8"	5	1667	928	566	1356
	21	20	3/8"	5	1607	854	579	1187
DI-1d	20	20	3/8"	5	1452	1114	562	1191
DI-2	18	20	3/8"	5	1500	1460	1216	1338
	20	20	3/8"	5	1286	1142	948	1077
	21	20	3/8"	5	882	786	545	796
DI-4	16	20	3/8"	4	2647	1230	570	2109
	18	20	3/8"	5	1667	1160	694	1315
	20	20	3/8"	5	726	668	442	722
	21	20	3/8"	5	804	656	390	719
DI-6	18	20	3/8"'	5	938	592	513	907
	20	20	3/8"'	5	381	376	324	565
	21	20	3/8"'	5	738	556	265	689

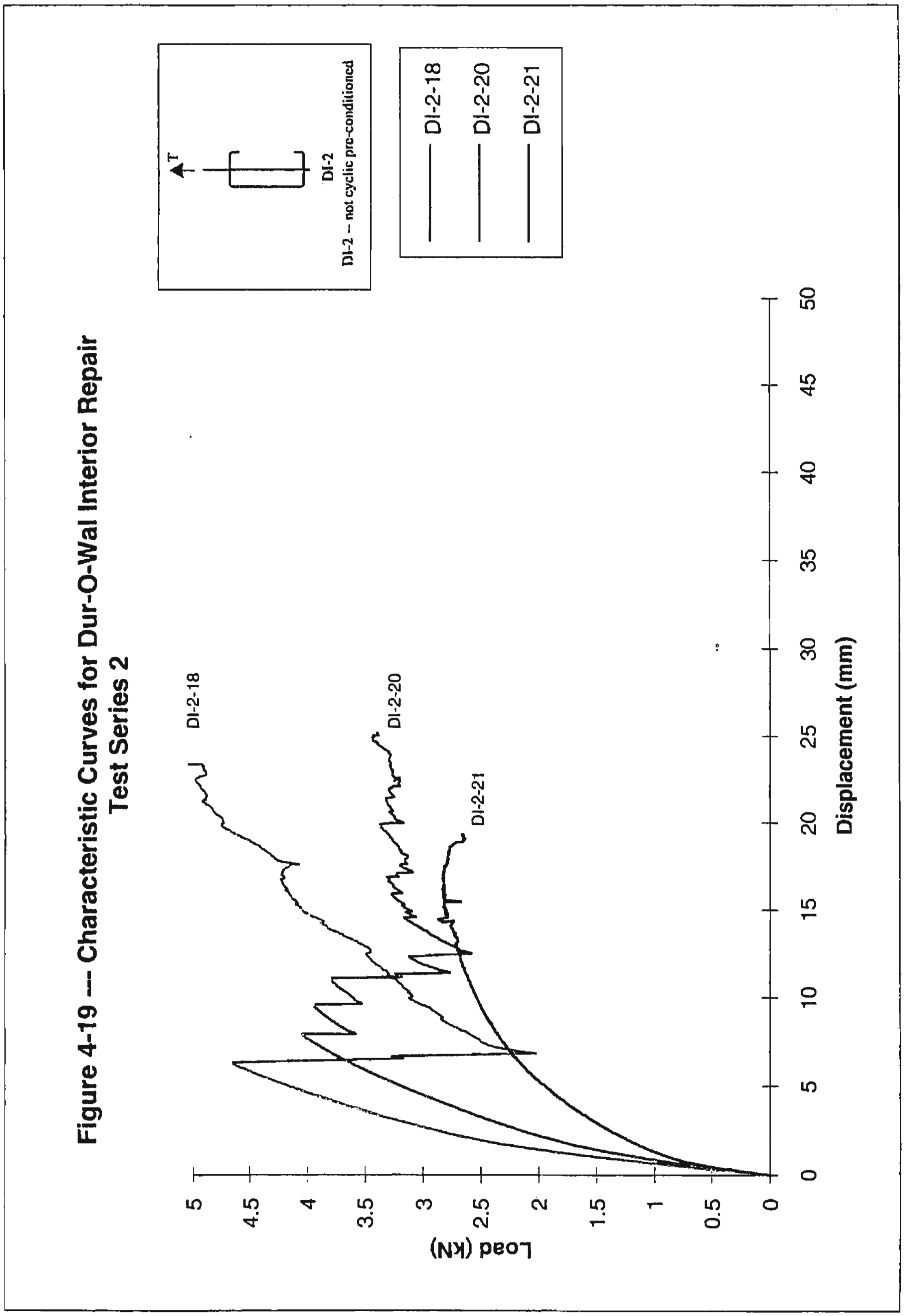


Figure 4-19 — Characteristic Curves for Dur-O-Wal Interior Repair Test Series 2

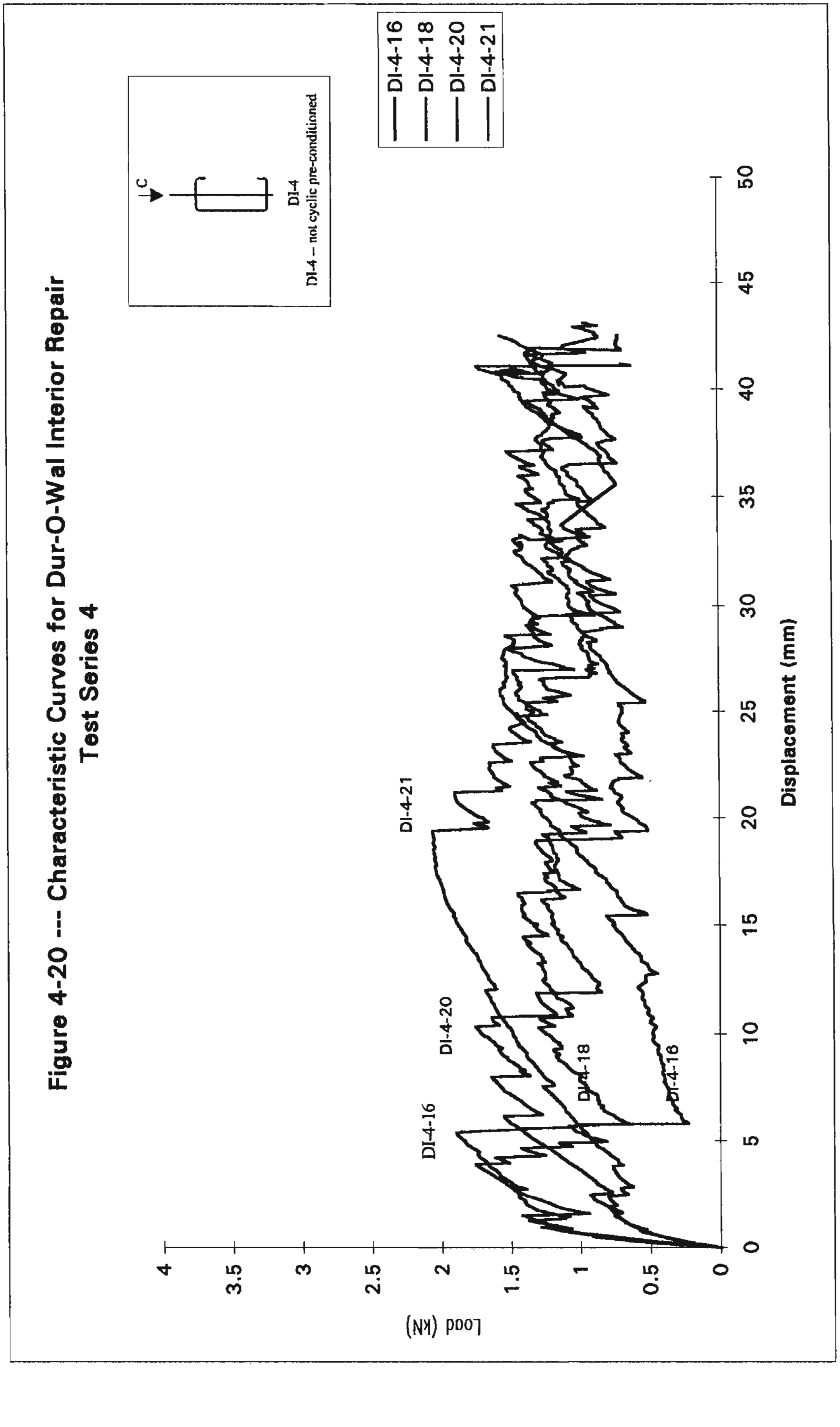


Figure 4-20 --- Characteristic Curves for Dur-O-Wal Interior Repair Test Series 4

4.6.3 Performance Under Cyclic Loading (Series 5 and 6)

Cyclic pre-conditioning was only conducted for ties in compression with 18, 20 and 21 gauge studs. Figure 4-21 shows the characteristic curves for compression with cyclic pre-conditioning for 18, 20 and 21 gauge studs. The effect of cyclic pre-conditioning was to decrease both the maximum load and the initial stiffness by approximately 20 to 30 %.

Displacement under cyclic preconditioning, illustrated in Figure 3-11, was recorded early in the cycling regime, at approximately 30 cycles and then again at 1000 cycles. Table 4-13 shows the amount of cyclic displacement for the Dur-O-Wal Interior tie with the different stud gauges and test setups. As is evident the amount of displacement recorded during cycling did not vary.

Table 4-13 - Displacements During Cyclic Loading for the Dur-O-Wal Interior Tie

	16 gauge		18 gauge		20 gauge		21 gauge	
	Isolation Tests mm	Beam Tests mm	Isolation Tests mm	Beam Tests mm	Isolation Tests mm	Beam Tests mm	Isolation Tests mm	Beam Tests mm
at 30 cycles	N.T.	N.T.	N.T.	0.11	0.06	0.29	N.T.	0.23
at 1000 cycles	N.T.	N.T.	N.T.	0.11	0.06	0.29	N.T.	0.23

N.T. - not tested

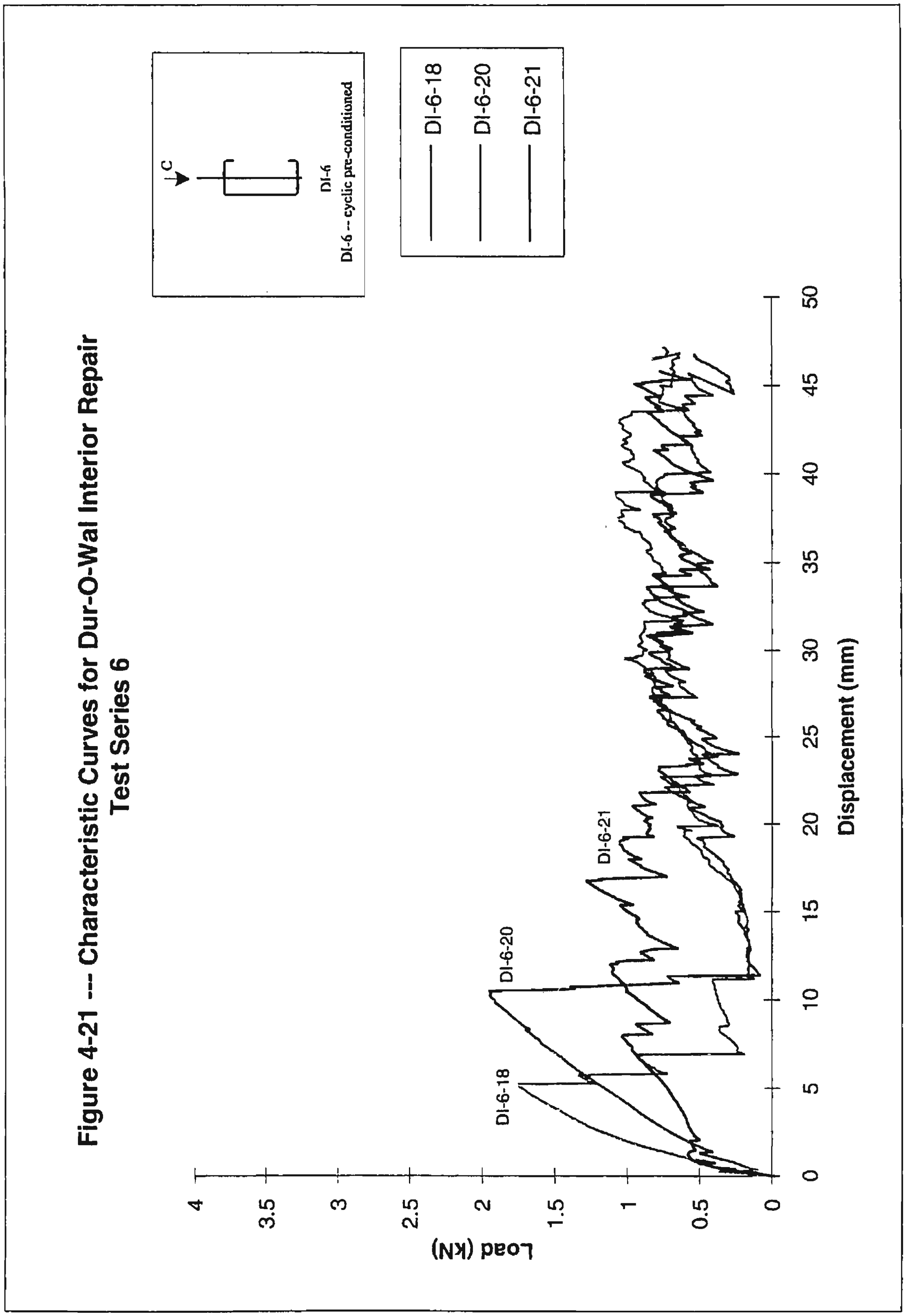


Figure 4-21 --- Characteristic Curves for Dur-O-Wal Interior Repair Test Series 6

4.6.4 Influence of Stud Gauge

The different stud gauges provided relatively the same values for maximum strength in compression. In both tension and compression the initial stiffness increased with increased stud thickness.

4.6.5 Influence of Attachment Location (Series 3)

No tests were conducted on the Dur-O-Wal Interior tie with different attachment locations on the flange of the steel stud. The performance of this tie will likely not be significantly altered by the location of the tie on the flange.

4.6.6 Comparison of Beam and Isolation Tests

All isolation tests for the Dur-O-Wal Interior tie were conducted in compression. The isolation tests exhibited greater stiffness than the beam tests in compression. Also much higher maximum compression loads were attained in the isolation tests, except with the 16 gauge stud where there was no difference.

5. DISCUSSION OF TEST RESULTS

5.1 Rigid Datum

The rigid datum tests provide the deformational characteristics for the steel studs alone under tension, compression and cyclic loading. Before discussing the performance of the repair systems it is important to consider how the steel stud deforms.

Figure 5-1 illustrates the deformation that occurs in the steel stud cross-section when a one flange connection is tested in either tension or compression. The initial stiffness of the two flange rigid datum connection in tension is as expected, much greater than that for the one flange connection. Figure 5-1 also shows the type of deformation that occurs in the two flange connection. There is little flange rotation and the cross-section does not deform as much as it does for the one flange connection.

When tested in tension the initial stiffness of the one flange rigid datum tie connection varied significantly with stud gauge. For the 20 and 21 gauge studs the one flange rigid datum connection exhibited similar stiffness values in tension and compression. However, at relatively high loads the response of the one flange rigid datum connection remains linear for tension while it becomes nonlinear under compression. The load at which this non-linearity occurs is approximately 0.75 kN for 21 gauge and 1.25 kN for 20 gauge.

From Table 4-1 the displacement at 0.45 kN for the one flange rigid datum connection was 0.25, 0.55, 1.1 and 1.6mm for 16, 18, 20 and 21 gauge studs respectively when loaded in tension. The compression values were similar to the tension values. The displacement for the 20 and 21 gauge studs are large in comparison to the 2mm at 0.45 kN serviceability criterion proposed in the new CAN/CSA A370. Although the exterior sheathing may reduce these displacements, a tie that attaches to the exterior flange only must have a stiff connection to meet this serviceability criterion. The resulting displacement in 16 and 18 gauge stud is less than with 20 and 21 gauge allowing a less stiff connection to be used in order to meet the criterion.

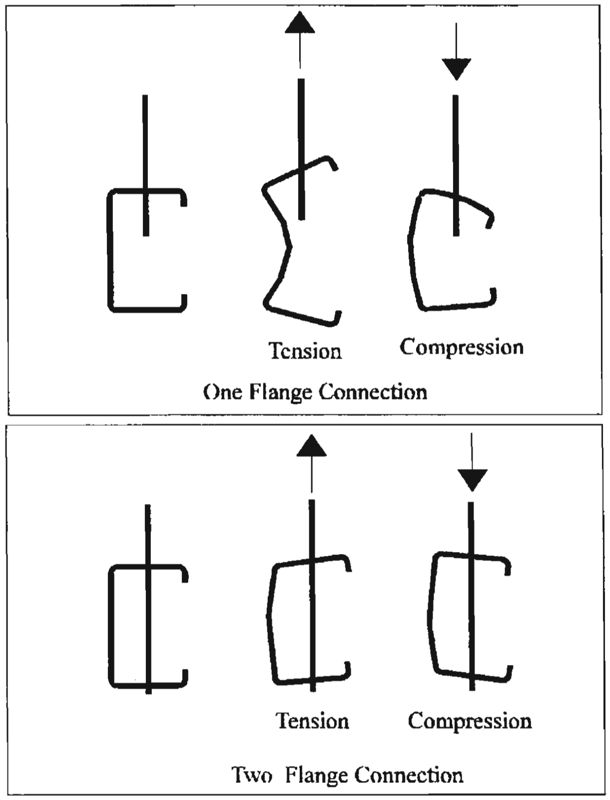


Figure 5-1 — Typical Deformations of Steel Stud Cross-Sections

In contrast the two flange rigid datum connection in 20 and 21 gauge stud only deflected 0.35 and 0.42 mm respectively under a load of 0.45 kN.

From a repair standpoint a tie installed in one 20 or 21 gauge flange only will need a repair tie connection that is relatively stiff in pullout characteristics if the serviceability criterion is to be met. For repairs in exterior flanges of heavier gauge or for repairs through two flanges in any of the gauges tested, a less stringent stiffness criterion on the tie to stud connection would be necessary.

The tests on the rigid datum ties also provided some interesting results on cyclic displacement. Under load cycles of 0.15 kN in tension and compression the amount of cyclic displacement was 0.11, 0.25, 0.6 and 0.52 for 16, 18, 20 , and 21 gauge respectively. This amount of cyclic deformation in the 20 and 21 gauge stud is not small. The 21 gauge had a lower cyclic displacement due to the smaller flange as compared to the 41mm flange of the 20 and heavier gauge studs.

5.2 Performance of Retrofit Ties

The performance of the four retrofit ties in steel stud will be discussed in terms of the general nature of their performance but with particular reference to the nature of failure and structural safety and structural serviceability.

5.2.1 Overall Response and the Nature of Failure

The nature of failure of a connection in a wall is not always considered an important performance parameter. However, the manner in which a tie fails is important as it gives an indication of the deformability, strength and stability of a connection. If a connection is required to undergo large displacements due to accidental or abnormal loadings such as an earthquake, impact or explosion, it is important for the designer to know the potential for sustained strength. The ability to absorb or shed energy and avoid restraint induced loads is a very important attribute. When a tie connection fails in a brittle manner irreversible damage has occurred and the rapidity of transfer may induce incremental collapse/failure. Ductile energy absorbing connections are, for example, highly advantageous to retrofit masonry in seismic areas. At this point serviceability is no longer of concern. Of concern is the ability of the connection system to accommodate abnormal loads without initiating a progressive failure. This ensures human safety and enables easier repair.

Perhaps the best way to illustrate the general significance of the nature of behaviour is to consider the eight figures that follow, i.e., the force-displacement relationships for each of the four types of retrofit tie (HE, DE, HI, DI) in both tension and compression for each of the four gauges of steel stud framing (16, 18, 20, 21). Where available the force-displacement relationship of the rigid datum (RD) is also shown. Significant aspects about the general response of these connections are as follows :

1. The Dur-O-Wal ties consistently have greater initial stiffness than the Helifix ties. In fact the initial stiffness of the Dur-O-Wal connections are for all practical purposes, initially the same as the Rigid Datum, i.e., all the movement is due to the stud and the tie does not undergo any relative movement. This is not the case with the Helifix ties where, it seems, the majority of the displacement is always provided by relative movement (free play) of the tie, even for 16 gauge studs.

2. Irrespective of stud gauge or direction of loading, the response of the Helifix tie is reasonably consistent. Ignoring the hump or peak when the tie in a one flange tie under compression touches the bottom flange, it is remarkable how consistent the response is. Note that, in general, the Helifix tie is weaker and perhaps less ductile under tension or outward loading.

3. The Dur-O-Wal exterior one flange connection is always less strong and less ductile, if not brittle, under tension which means that tension (pullout) rather than push-in is the critical direction.

4. The Dur-O-Wal interior fix, with a two flange connection, is always less strong and less ductile in compression. This fix undergoes a significant and steep loss of resistance when the epoxy starts spalling. Although failure, as such, does not occur the drop in resistance is so significant that it should be considered the limit on useful life.

These four conclusions effectively characterize the nature of the performance of each of these retrofit fixes. It remains to establish the relevance of these characteristics and to quantify the usable features of each retrofit tie solution.

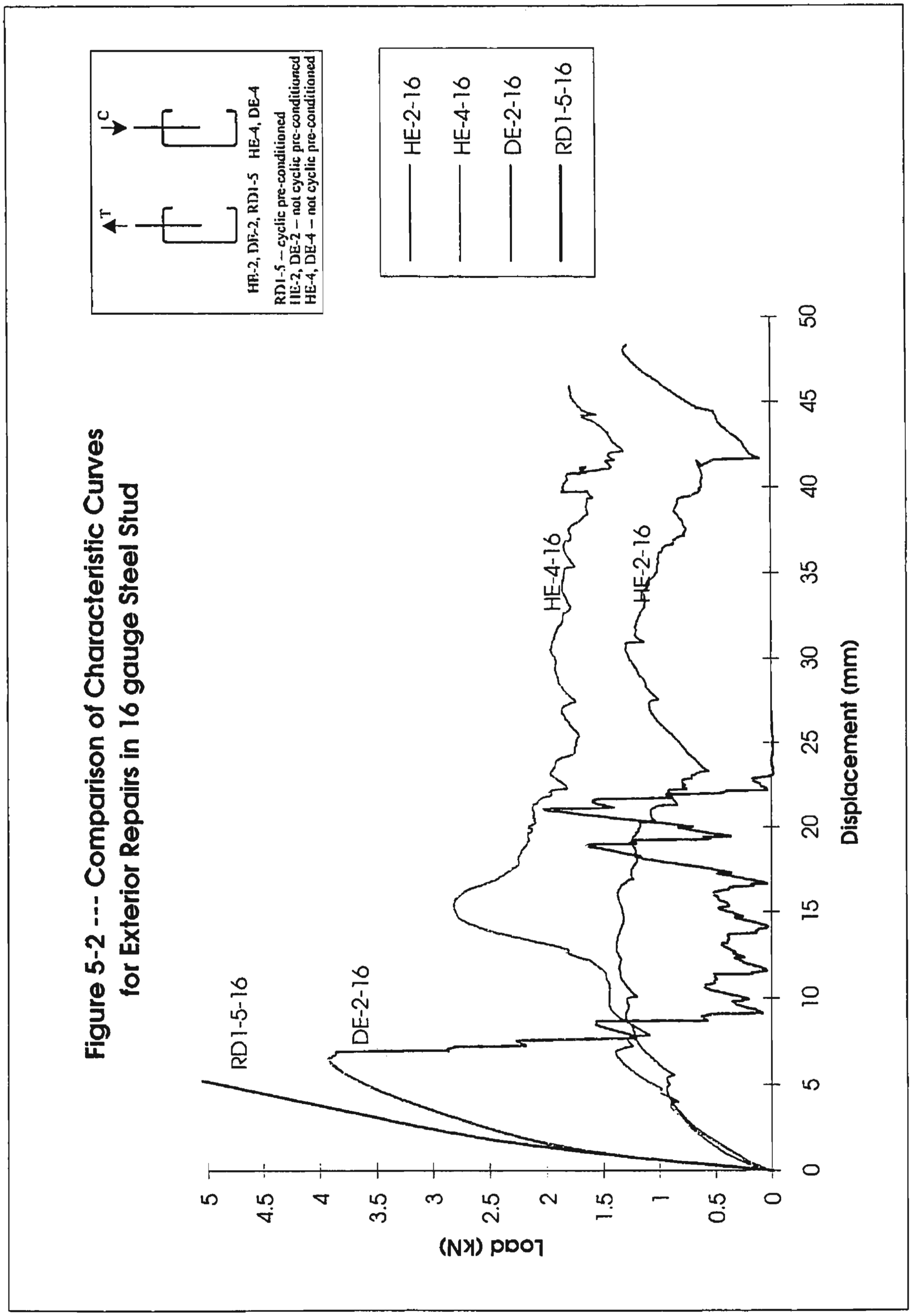


Figure 5-2 --- Comparison of Characteristic Curves for Exterior Repairs in 16 gauge Steel Stud

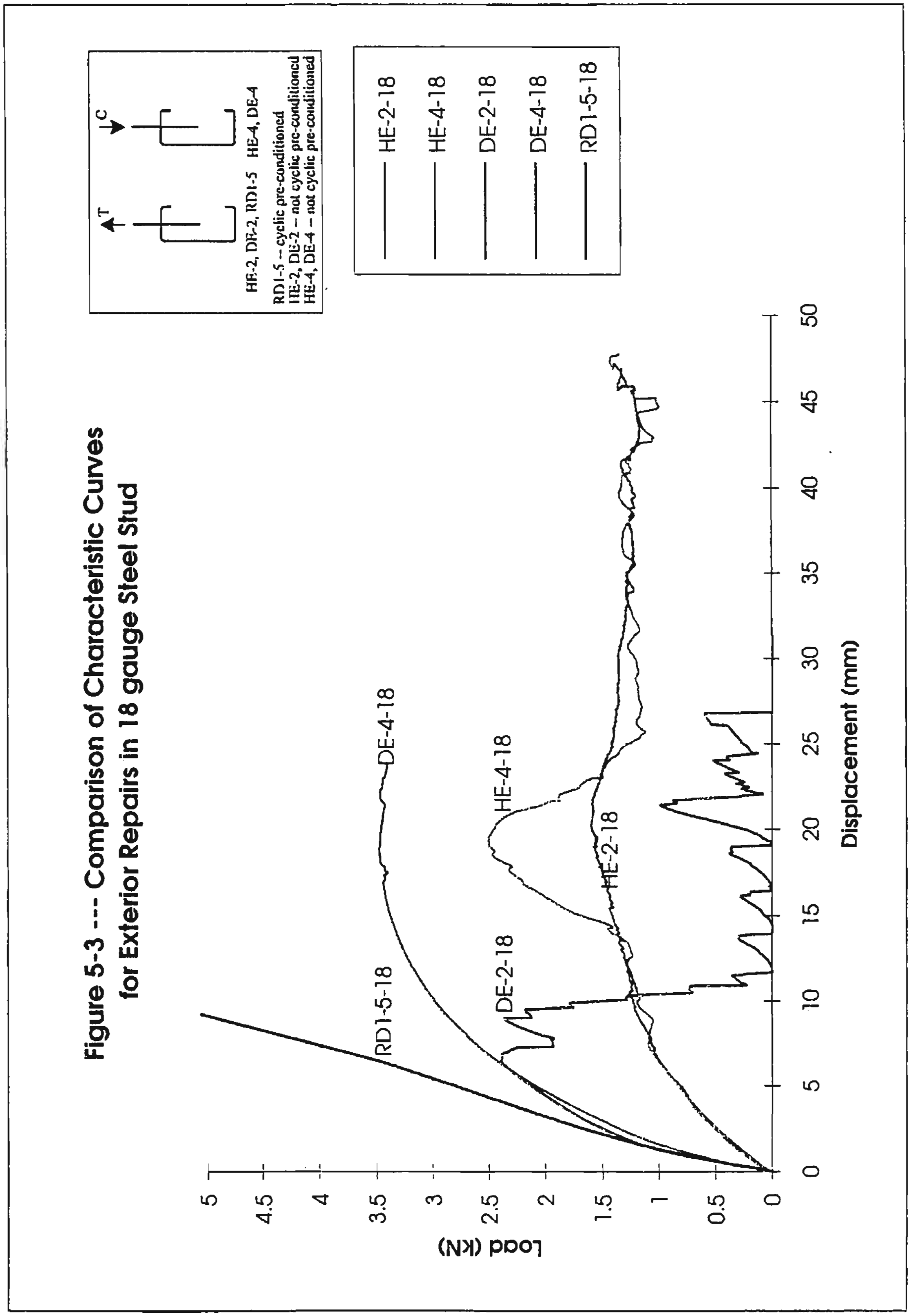


Figure 5-3 --- Comparison of Characteristic Curves for Exterior Repairs in 18 gauge Steel Stud

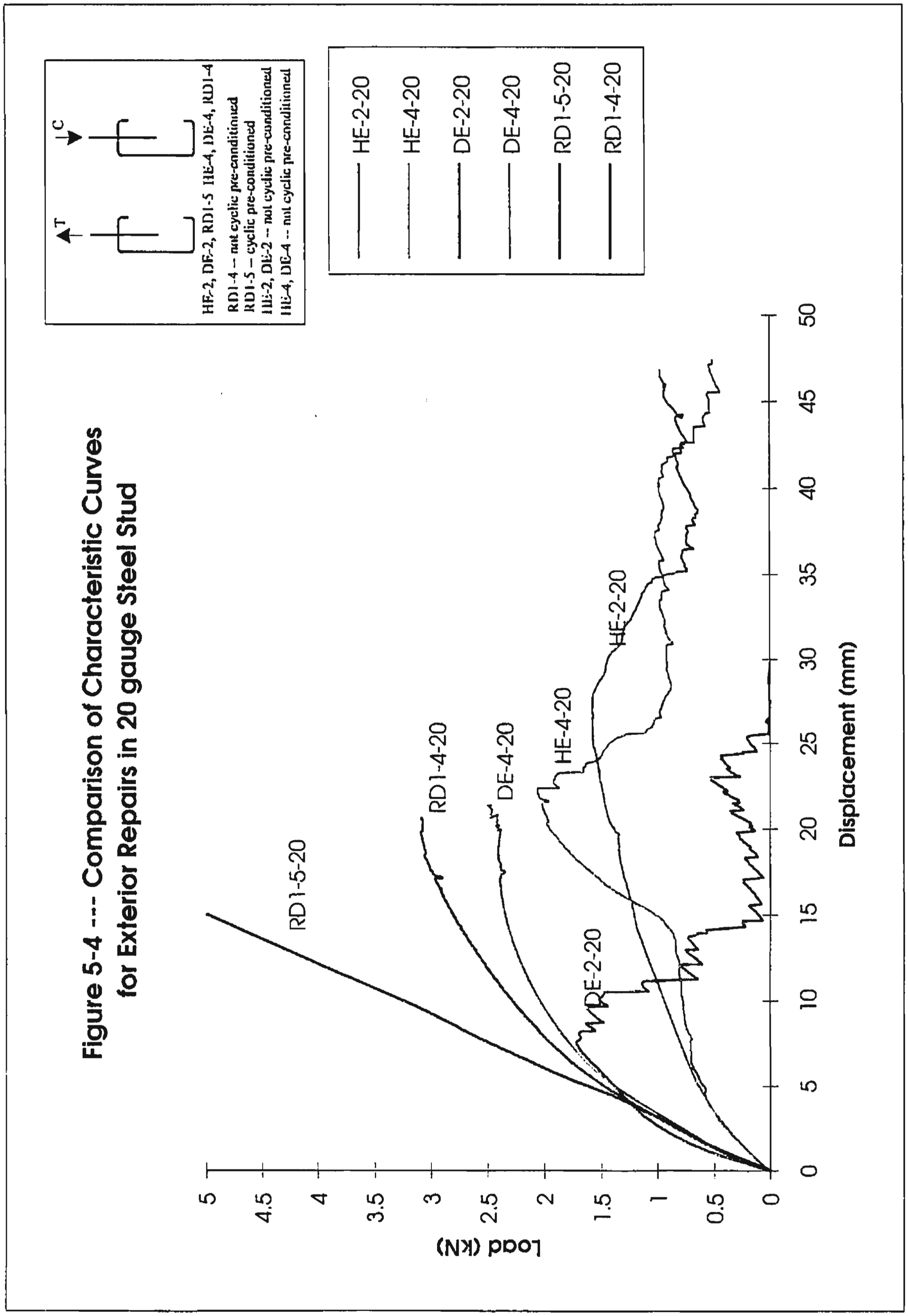


Figure 5-4 --- Comparison of Characteristic Curves for Exterior Repairs in 20 gauge Steel Stud

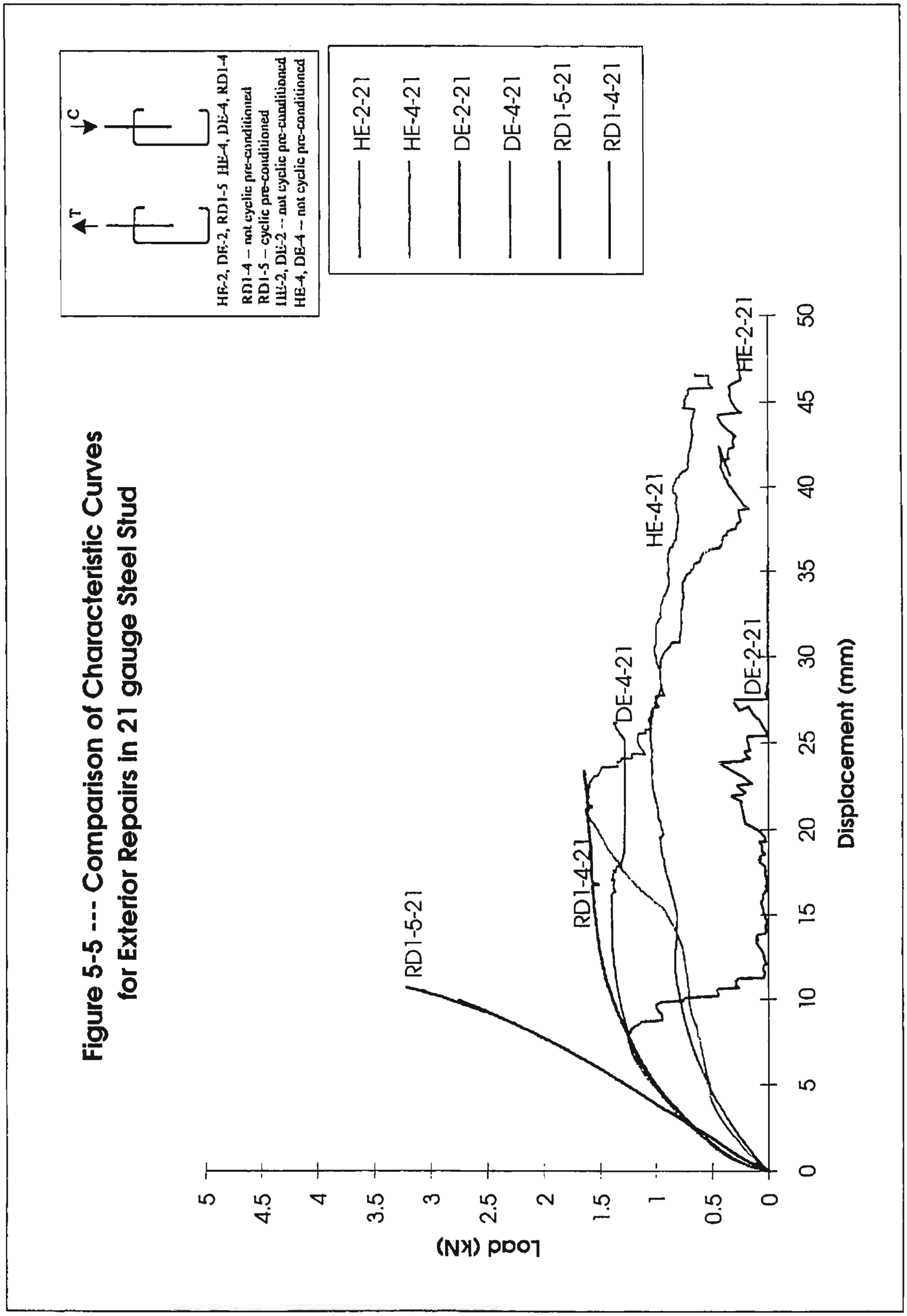


Figure 5-5 --- Comparison of Characteristic Curves for Exterior Repairs in 21 gauge Steel Stud

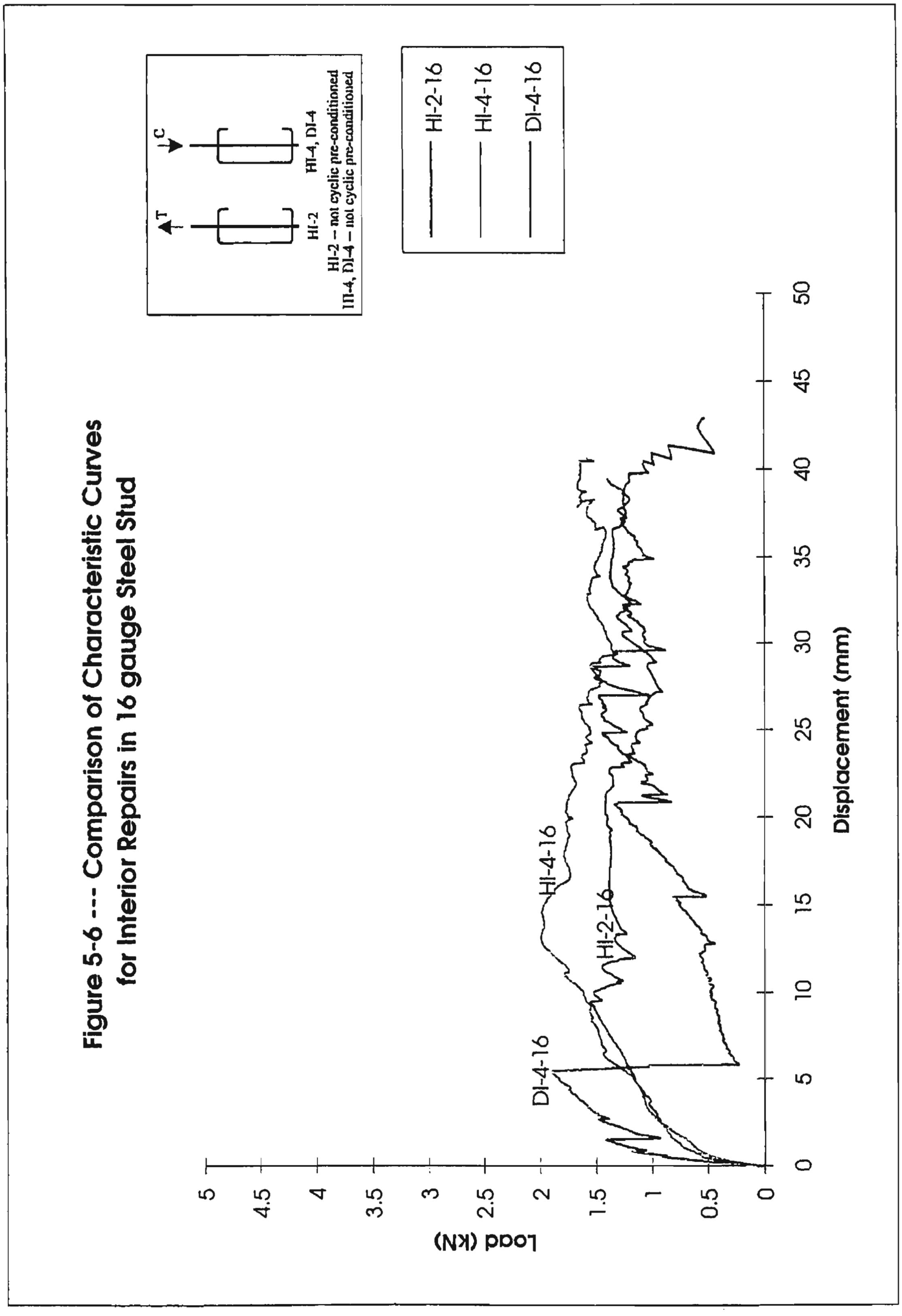


Figure 5-6 --- Comparison of Characteristic Curves for Interior Repairs in 16 gauge Steel Stud

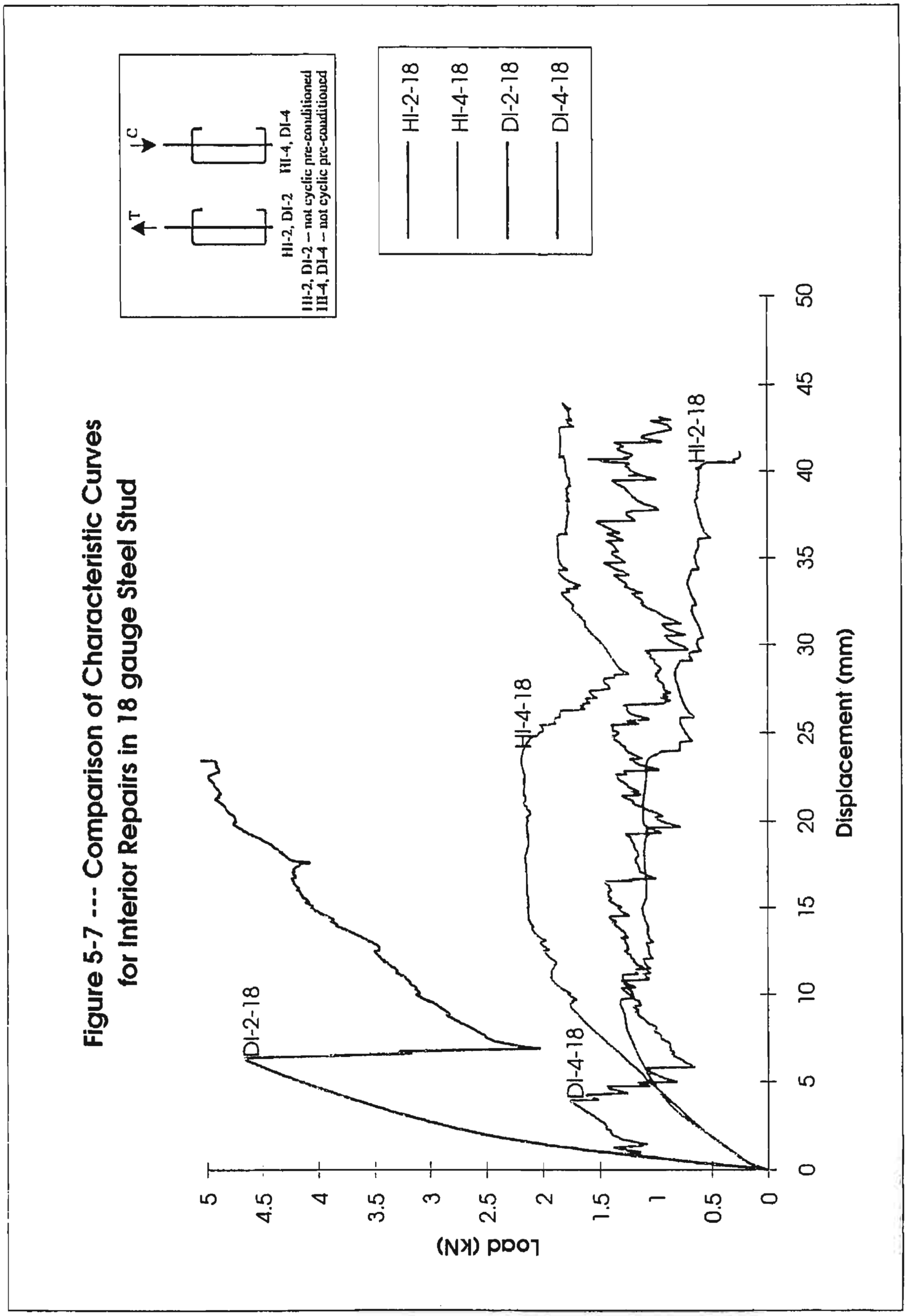


Figure 5-7 --- Comparison of Characteristic Curves for Interior Repairs in 18 gauge Steel Stud

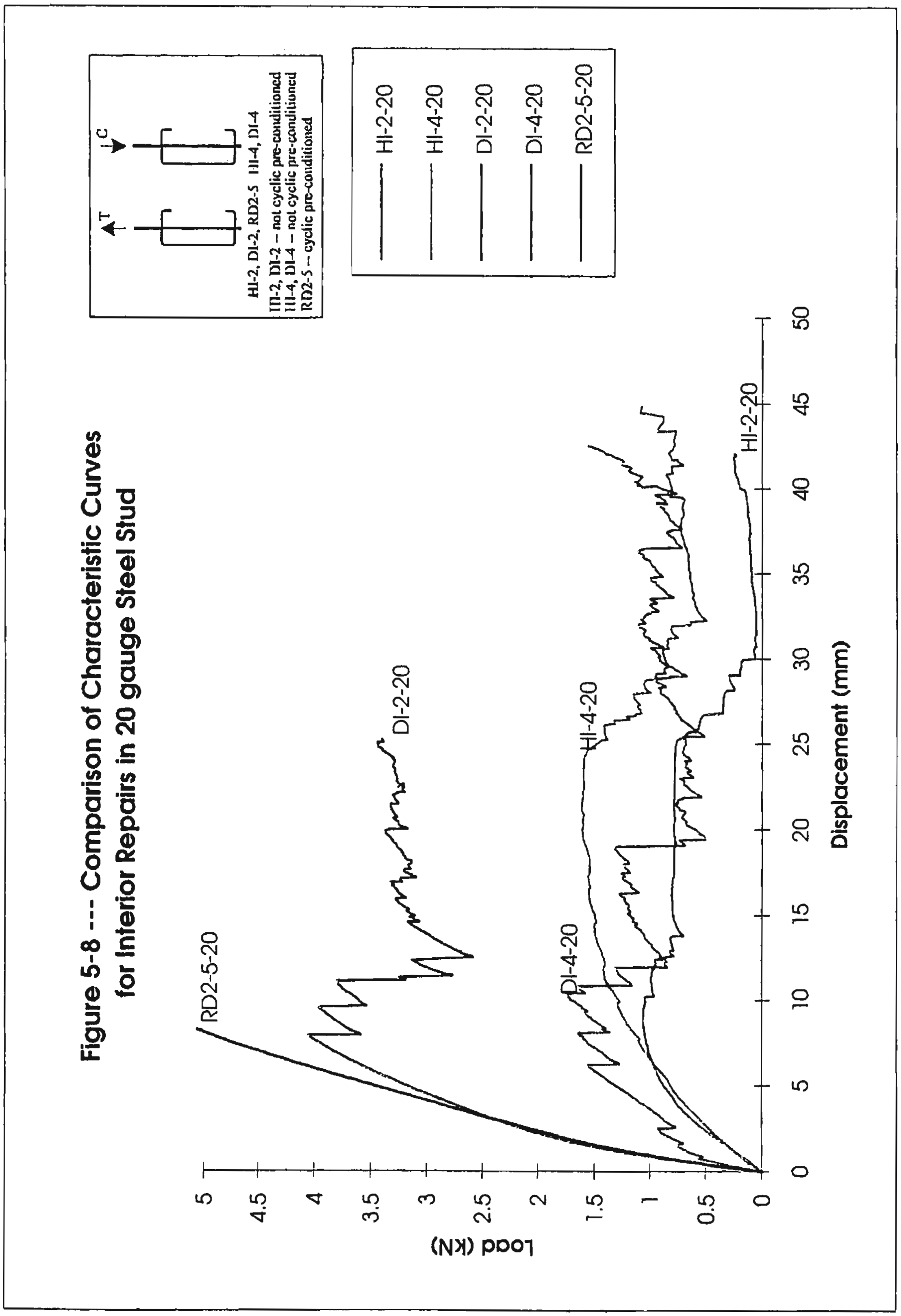


Figure 5-8 --- Comparison of Characteristic Curves for Interior Repairs in 20 gauge Steel Stud

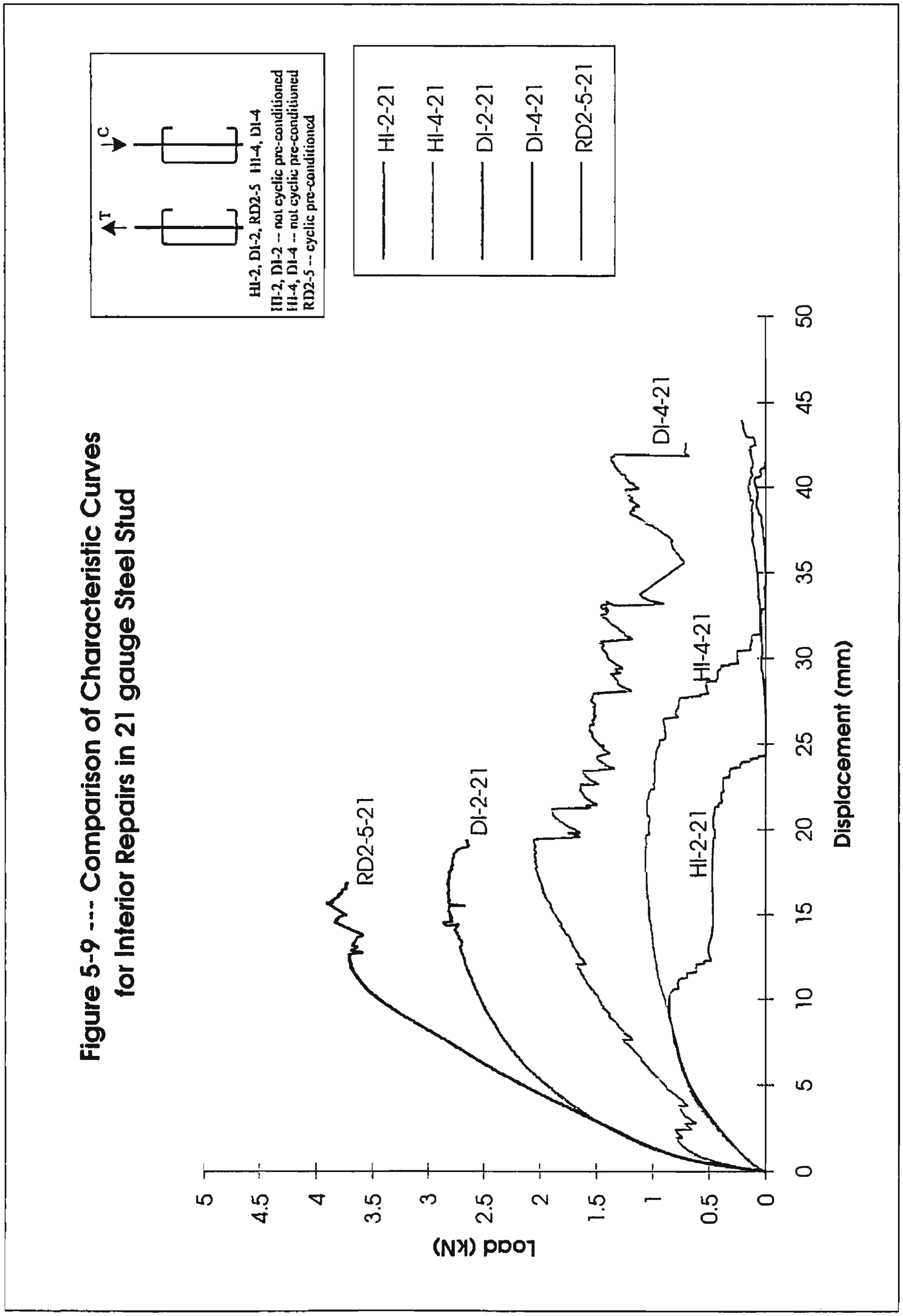


Figure 5-9 --- Comparison of Characteristic Curves for Interior Repairs in 21 gauge Steel Stud

5.2.2 Structural Safety

The structural safety of the various retrofit ties will be discussed with respect to the requirements specified in CSA/CAN A370-93 and outlined in Chapter 2. This code has provisions for both the Working Stress Design and Limit States Design approach. Table 2-4 summarized the two different methods. The two governing criteria were :

for WSD, $\frac{R_{char}}{F.S.} \geq S_w$ & for LSD, $\frac{\phi}{\alpha_Q} R_{char} \geq S_W$

and considering the typical failure condition of embedment the factor of safety for WSD and the resistance factors for LSD are $F.S. = 4.0$, $\alpha_Q = 1.5$, $\phi = 0.55$

for WSD, $\frac{R_{char}}{4} \geq S_w$ & for LSD, $\frac{R_{char}}{2.73} \geq S_w$

The LSD provisions reflect a lower overall factor of safety than the WSD method. For the purpose of discussion only the new limit states method of calculating the resistance will be considered; largely because the WSD approach will be dropped in future codes.

The characteristic value is obtained from maximum values from test results as follows :

$$R_{char} = \bar{x} - 1.5S.D.$$

This value is representative of a confidence level of 93.32 %. The maximum permissible design value at the service load level is :

$$R_w = \frac{R_{char}}{2.73}$$

While this approach is the one recommended in the code it is worth assessing the statistical merits of these values. The equivalent number of standard deviations indicates the probability of whether the maximum strength of a tie will be less than the factored resistance given for the LSD method. The high variability in the equivalent number of standard deviations indicates large variations in the actual value that results. To obtain a more consistent reserve a specific failure rate or consistent confidence interval could be adopted. For example, a failure rate such as every 1 in 10000 ties. The resistance for each test series can be calculated using

$$R = \bar{x} + zS.D.$$

where

R - resistance or response parameter
$\bar{x}$ - average of test results
$S.D.$ - standard deviation of test results
z - normal deviate value

The normal deviate, z, is constant for a particular failure rate. Table 5-1 lists normal deviate values for different failure rates.

Table 5-1 - Normal Deviate Values for Failure Rates

Failure Rate	Normal Deviate, z
1 in 100	-2.33
1 in 1000	-3.1
1 in 5000	-3.5
1 in 10000	-3.8
1 in >10000	< -3.8

The factored resistance for a failure rate of 1 in 10000 can be calculated with a z value of -3.8. This factored resistance has the statistical meaning that only 1 in every 10000 ties will have a maximum strength less than the factored resistance value.

There are three fundamental reasons why the LSD method is preferable to the failure criteria method for calculating factors of safety. First the failure criterion method does not explicitly account for deficiencies in workmanship or installation practices. If one considers one in every two ties to have zero resistance and divide the 1 in 10000 factored resistance by 2 the result is a value that is close to the LSD method. The problem in masonry construction is that, when workmanship problems occur, it is not likely to be only one tie but a number of adjacent ties. The second reason that the LSD method is preferable to a statistical approach is that the factored resistance is determined for a very isolated case in the laboratory. These laboratory tests are typically based on one test setup with well controlled material properties and installation procedures. The variability of these properties in an actual wall is difficult to consider. The third reason is the number of tests conducted. As testing can be expensive a minimum of five tests is typically conducted. For this number of specimens the use of the normal deviate approach is questionable, as the standard deviation can be relatively large.

Tables 5-2, 5-3, 5-4 and 5-5 list the characteristic strength values for each test series.

From the numerous test series the governing values for the characteristic strength, R_{char}, and the service level resistance, R_w, were determined. The governing values were typically taken as the lowest value for each tie in each gauge. All test series, except Test Series 3 were considered to determine the governing value. In some cases some judgment was used to select a higher value if the lowest value had a large standard deviation. Table 5-6 contains the governing values of the characteristic strengths while Figure 5-10 illustrates these values graphically. Table 5-7 contains the governing values of the factored resistance using the LSD factor of safety of 2.73. Also listed in Table 5-7 is the recommended design resistance for each tie in the four gauges of steel stud.

Before identifying a recommended design value there is some merit in examining these values a bit more carefully. Firstly it will be noticed that the value of the S.D. becomes a very important consideration - for instance in all those cases where the S.D. is greater than $\bar{x}/5$ this value is identified by shading or darkening of the value in the Tables. It is apparent that the greatest variability with regard to maximum capacity occurs with the Dur-O-Wal interior repair. Clearly this has a significant effect on the values for the characteristic strength and illustrates why the values in Table 5-6 for this connection are not consistent. In fact the mean values for DI, in compression, range between 2.19 and 2.63 with the 16 gauge stud giving the largest mean value but also the largest standard deviation. To base a design value on this spread of characteristics strengths is not really appropriate.

TABLE 5-2 -- Characteristic Strengths and Resistances for the Helifix Exterior Repair

1 Test Series	2 Average Maximum Load $\bar{x}$ kN	3 Standard Deviation $S.D.$ kN	4 Characteristic Strength R_{char} kN	5 Resistance for F.S. = 2.73 LSD R_w kN	6 Equivalent Number of Standard Deviations n F.S. = 2.73	7 1 in 10000 Resistance R_{10000} kN
HE-1a-20	1.64	0.13	1.445	0.529	8.54	1.146
HE-1b-16*	1.52	0.07	1.415	0.518	14.31	1.254
HE-1b-18*	1.23	0.11	**1.065**	**0.390**	7.64	0.812
HE-1b-20*	0.87	0.06	0.780	0.286	9.74	0.642
HE-1b-21*	0.61	0.06	**0.520**	**0.190**	6.99	0.382
HE-1bA-20	0.92	0.04	0.860	0.315	15.12	0.768
HE-1c-16	1.72	0.09	1.585	0.581	12.66	1.378
HE-1c-18	1.76	0.11	1.595	0.584	10.69	1.342
HE-1c-20	1.72	0.20	1.420	0.520	6.00	0.960
HE-1c-21	1.03	0.06	0.940	0.344	11.43	0.802
HE-1d-20*	0.93	0.06	0.840	0.308	10.37	0.702
HE-2-16	1.58	0.13	1.385	0.507	8.25	**1.086**
HE-2-18	1.66	0.19	1.375	0.504	6.09	0.938
HE-2-20	1.65	0.16	1.410	0.516	7.08	1.042
HE-2-21	1.11	0.04	1.050	0.385	18.13	0.958
HE-3-10	1.84	0.11	1.675	0.614	11.15	1.422
HE-3-20	1.65	0.16	1.410	0.516	7.08	1.042
HE-3-30	1.70	0.21	1.385	0.507	5.68	0.902
HE-4-16*	1.51	0.11	**1.345**	**0.493**	9.25	1.092
HE-4-18*	1.34	0.11	1.175	0.430	8.27	0.922
HE-4-20*	0.88	0.10	**0.730**	**0.267**	6.13	0.500
HE-4-21*	0.79	0.13	0.595	0.218	4.40	**0.000**
HE-4A-20	0.98	0.03	0.935	0.342	21.25	0.866
HE-5-18	1.88	0.35	1.355	0.496	3.95	**0.550**
HE-5-20	1.72	0.24	1.360	0.498	5.09	0.808
HE-5-21	1.04	0.09	0.905	0.332	7.87	0.698
HE-6-20*	0.92	0.12	0.740	0.271	5.41	**0.464**

$$\boxed{n = \frac{\bar{x} - R}{S.D.}}$$

$$\boxed{R_{10000} = \bar{x} - 3.8S.D.}$$

Shaded cells indicate the standard deviations of the tests which have a coefficient of variation greater than 20 %

BOLD values are the governing values for that particular gauge of stud

* the maximum load in these series is the load immediately before contact with the inner flange

TABLE 5-3 -- Characteristic Strengths and Resistances for the Dur-O-Wal Exterior Repair

1 Test Series	2 Average Maximum Load $\bar{x}$ kN	3 Standard Deviation $S.D.$ kN	4 Characteristic Strength R_{char} kN	5 Resistance for F.S. = 2.73 LSD R_w kN	6 Equivalent Number of Standard Deviations n F.S. = 2.73	7 1 in 10000 Resistance R_{10000} kN
DE-1a-20	2.57	0.15	2.345	0.859	11.41	2.000
DE-1b-20	5.00	0.00	5.000	1.832	N.A.	5.000
DE-1c-16	4.33	0.19	4.045	1.482	14.99	3.608
DE-1c-18	3.22	0.27	2.815	1.031	8.11	2.194
DE-1c-20	2.43	0.06	2.340	0.857	26.21	2.202
DE-1c-21	1.75	0.12	1.570	0.575	9.79	1.294
DE-2-16	4.05	0.28	**3.630**	**1.330**	9.72	**2.986**
DE-2-18	2.67	0.27	2.265	0.830	6.82	1.644
DE-2-20	2.17	0.25	1.795	0.658	6.05	1.220
DE-2-21	1.37	0.05	1.295	0.474	17.91	1.180
DE-3-10	2.36	0.20	2.060	0.755	8.03	1.600
DE-3-20	2.17	0.25	1.795	0.658	6.05	1.220
DE-3-30	2.02	0.16	1.780	0.652	8.55	1.412
DE-4-18	3.54	0.11	3.375	1.236	20.94	3.122
DE-4-20	2.42	0.12	2.240	0.821	13.33	1.964
DE-4-21	1.42	0.05	1.345	0.493	18.55	1.230
DE-5-18	2.68	0.35	**2.155**	**0.789**	5.40	**1.350**
DE-5-20	2.06	0.26	**1.670**	**0.612**	5.57	**1.072**
DE-5-21	1.34	0.10	**1.190**	**0.436**	9.04	**0.960**
DE-6-20	2.61	0.15	2.385	0.874	11.58	2.040

$$n = \frac{\bar{x} - R}{S.D.}$$

$$R_{10000} = \bar{x} - 3.8 S.D.$$

Shaded cells indicate the standard deviations of the tests which have a coefficient of variation greater than 20 %

BOLD values are the governing values for that particular gauge of stud

TABLE 5-4 -- Characteristic Strengths and Resistances for the Helifix Interior Repair

1 Test Series	2 Average Maximum Load $\bar{x}$ kN	3 Standard Deviation $S.D.$ kN	4 Characteristic Strength R_{char} kN	5 Resistance for F.S. = 2.73 LSD R_w kN	6 Equivalent Number of Standard Deviations n F.S. = 2.73	7 1 in 10000 Resistance R_{10000} kN
HI-1a-20	1.21	0.13	1.015	0.372	6.45	**0.716**
HI-1b-16	2.05	0.57	1.195	0.438	2.83	**0.000**
HI-1b-18	2.22	0.07	2.115	0.775	20.65	1.954
HI-1b-20	1.75	0.08	1.630	0.597	14.41	1.446
HI-1b-21	1.08	0.07	0.975	0.357	10.33	0.814
HI-1c-16	1.86	0.53	1.065	0.390	2.77	0.000
HI-1c-18	1.77	0.22	1.440	0.527	5.65	0.934
HI-1c-20	1.23	0.08	1.110	0.407	10.29	0.926
HI-1c-21	0.85	0.04	0.790	0.289	14.02	0.698
HI-1d-20	1.71	0.05	1.635	0.599	22.22	1.520
HI-2-16	1.78	0.26	**1.390**	**0.509**	4.89	**0.792**
HI-2-18	1.39	0.19	**1.105**	**0.405**	5.19	0.668
HI-2-20	1.10	0.05	**1.025**	**0.375**	14.49	0.910
HI-2-21	0.88	0.09	**0.745**	**0.273**	6.75	**0.538**
HI-3-10	1.22	0.21	0.905	0.332	4.23	0.422
HI-3-20	1.10	0.05	1.025	0.375	14.49	0.910
HI-3-30	1.13	0.17	0.875	0.321	4.76	0.484
HI-4-16	2.08	0.26	1.690	0.619	5.62	1.092
HI-4-18	2.22	0.05	2.145	0.786	28.69	2.030
HI-4-20	1.65	0.19	1.365	0.500	6.05	0.928
HI-4-21	1.10	0.04	1.040	0.381	17.98	0.948
HI-5-18	1.44	0.29	1.005	0.368	3.70	0.338
HI-5-20	1.16	0.05	1.085	0.397	15.25	0.970
HI-5-21	0.92	0.06	0.830	0.304	10.27	0.692
HI-6-20	1.65	0.11	1.485	0.544	10.05	1.232

$$\boxed{n = \frac{\bar{x} - R}{S.D.}}$$

$$\boxed{R_{10000} = \bar{x} - 3.8S.D.}$$

Shaded cells indicate the standard deviations of the tests which have a coefficient of variation greater than 20 %

BOLD values are the governing values for that particular gauge of stud

TABLE 5-5 -- Characteristic Strengths and Resistances for the Dur-O-Wal Interior Repair

1 Test Series	2 Average Maximum Load $\bar{x}$ kN	3 Standard Deviation $S.D.$ kN	4 Characteristic Strength R_{char} kN	5 Resistance for F.S. = 2.73 LSD R_w kN	6 Equivalent Number of Standard Deviations n F.S. = 2.73	7 1 in 10000 Resistance R_{10000} kN
DI-1b-16	2.63	0.93	1.235	0.452	2.34	0.000
DI-1b-18	3.31	0.69	2.275	0.833	3.59	**0.688**
DI-1b-20	4.57	0.73	3.475	1.273	4.52	1.796
DI-1b-21	2.98	0.22	2.650	0.971	9.13	2.144
DI-1d-20	4.82	0.23	4.475	1.639	13.83	3.946
DI-2-18	5.16	0.69	4.125	1.511	5.29	2.538
DI-2-20	4.45	0.33	3.955	1.449	9.09	3.196
DI-2-21	2.53	0.44	1.870	0.685	4.19	0.858
DI-4-16	2.63	0.93	**1.235**	**0.452**	2.34	**0.000**
DI-4-18	2.20	0.28	1.780	0.652	5.53	1.136
DI-4-20	2.33	0.46	1.640	0.601	3.76	0.582
DI-4-21	2.19	0.06	2.100	0.769	23.68	1.962
DI-6-18	1.87	0.31	**1.405**	**0.515**	4.37	0.692
DI-6-20	1.83	0.39	**1.245**	**0.456**	3.52	**0.348**
DI-6-21	1.57	0.38	**1.000**	**0.366**	3.17	**0.126**

$$n = \frac{\bar{x} - R}{S.D.}$$

$$R_{10000} = \bar{x} - 3.8S.D.$$

Shaded cells indicate the standard deviations of the tests which have a coefficient of variation greater than 20 %

BOLD values are the governing values for that particular gauge of stud

Table 5-6 Governing Characteristic Strengths

Gauge	Helifix Exterior Tie R_{char} kN	Dur-O-Wal Exterior Tie R_{char} kN	Helifix Interior Tie R_{char} kN	Dur-O-Wal Interior Tie R_{char} kN
16	1.345	3.630	1.390	1.235
18	1.065	2.155	1.105	1.405
20	0.730	1.670	1.025	1.245
21	0.520	1.190	0.745	1.000

Figure 5-10 --- Governing Characteristic Strengths

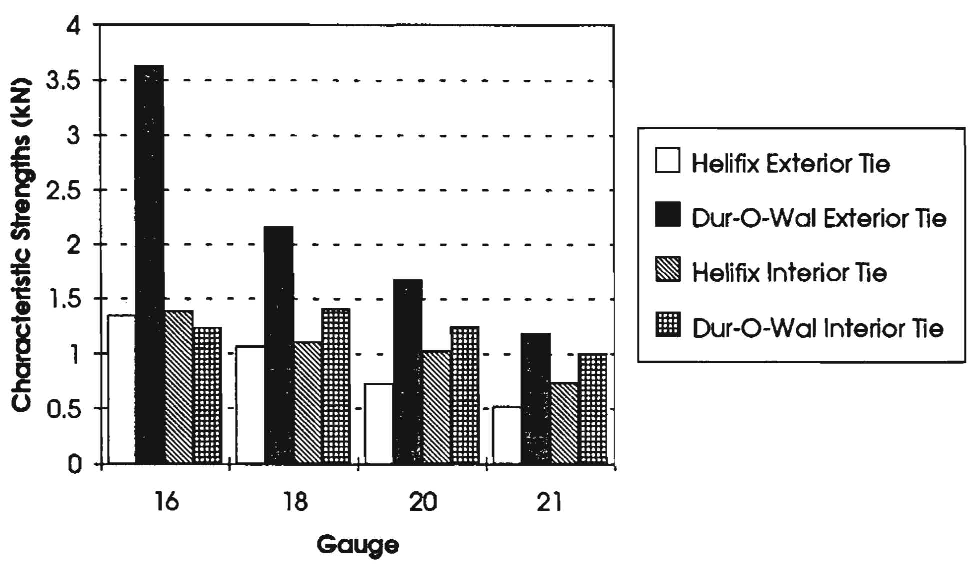


Table 5-7 -- Governing Resistances and Recommended Design Values

Governing Resistances

	Helifix Exterior Tie		Dur-O-Wal Exterior Tie		Helifix Interior Tie		Dur-O-Wal Interior Tie	
	R_w kN	Hole Dia. inches	R_w kN	Hole Dia. inches	R_w kN	Hole Dia. inches	R_w kN	Hole Dia. inches
16	0.493	1/4"	1.330	3/16"	0.509	1/4"	0.452	3/8"
18	0.390	1/8"	0.789	5/32"	0.405	1/4"	0.515	3/8"
20	0.267	3/32"	0.612	5/32"	0.375	1/4"	0.456	3/8"
21	0.190	3/32"	0.436	5/32"	0.273	1/4"	0.366	3/8"

Recommended Design Values

	Helifix Exterior Tie		Dur-O-Wal Exterior Tie		Helifix Interior Tie		Dur-O-Wal Interior Tie	
	R_w kN	Hole Dia. inches	R_w kN	Hole Dia. inches	R_w kN	Hole Dia. inches	R_w kN	Hole Dia. inches
16	0.50	1/4"	1.30	3/16"	0.50	1/4"	0.45	3/8"
18	0.40	1/8"	0.77	5/32"	0.40	1/4"	0.45	3/8"
20	0.25	3/32"	0.60	5/32"	0.35	1/4"	0.45	3/8"
21	0.20	3/32"	0.43	5/32"	0.27	1/4"	0.36	3/8"

Note : hole diameters are in inch units because the drill bit sizes were Imperial

Secondly it is possible to assess the factored resistance value as follows i.e., $R = \bar{x} + zS.D.$ or $n = \dfrac{\bar{x} - R}{S.D.}$. Values for n are shown in column 6 and it is evident that the value for n is not consistent, which clearly indicates that with respect to R the confidence level is inconsistent and highly variable. It follows that an alternative approach to establishing values for R would be to attempt to establish a consistent confidence interval say 1 in 10000 or a value of n of -3.8. These values are listed in column 7.

5.2.3 Structural Serviceability

The serviceability of the various retrofit ties will be discussed with respect to the requirements specified in CSA/CAN A370-93 as outlined in Section 2.

Table 5-8 contains the cyclic displacement values for the four different connections and the rigid datum connection. Note that in some test series there was no cyclic pre-conditioning. The cyclic displacement was recorded at 30 and at 1000 cycles for each of the test series.

The cyclic displacement at 1000 cycles was generally smaller than at 30 cycles. This is due to the working of the steel around the connection. With repetitive loading the Helifix tie firmly seats itself in the stud flange and cyclic displacements decrease. The cyclic displacement values for the two Dur-O-Wal ties and the rigid datum situation did not change over the period of 1000 cycles.

Figure 5-11 graphically shows the cyclic displacement of the ties at 1000 cycles for the beam tests. This figure clearly shows that the Dur-O-Wal Exterior tie has the same cyclic displacement as the rigid one flange connection. The Helifix Exterior tie has for the different gauges a 30 to 100% increase in cyclic displacement over the rigid one flange connection. The cyclic displacement for the Dur-O-Wal Exterior tie consists of flange rotation as with the rigid connection and no slip in the actual connection occurs. For the Helifix tie, the cyclic displacement includes the flange rotation but there is also displacement occurring at the tie to flange connection. This displacement includes tie slip in the pre-drilled hole and local deformation of the steel around the flanges of the tie.

The Dur-O-Wal Interior tie and the Helifix Interior tie show higher displacements than the two flange rigid datum connection. The Helifix tie again has greater cyclic displacement than the Dur-O-Wal tie. The displacement of the Helifix Interior tie includes flange rotation, tie slip and local deformation of the steel around the tie. The Helifix Interior tie which engages both flanges of the steel stud has lower cyclic displacements than the Helifix Exterior tie. The amount of cyclic displacement in the Helifix Interior tie is very similar to the Dur-O-Wal Exterior tie.

Table 5-8 --- Summary of Cyclic Displacements

	16 gauge		18 gauge		20 gauge		21 gauge	
	Isolation Tests mm	Beam Tests mm	Isolation Tests mm	Beam Tests mm	Isolation Tests mm	Beam Tests mm	Isolation Tests mm	Beam Tests mm
Helifix Exterior Tie								
at 30 cycles	0.69	N.T.	0.2	0.76	0.7	1.06	1.13	1.55
at 1000 cycles	0.22	N.T.	0.12	0.49	0.24	0.76	0.7	1.31
Dur-O-Wal Exterior Tie								
at 30 cycles	0.03	N.T.	0.08	0.27	0.16	0.57	0.26	0.49
at 1000 cycles	0.03	N.T.	0.07	0.27	0.14	0.57	0.24	0.49
Helifix Interior Tie								
at 30 cycles	0.02	N.T.	0.11	0.41	0.47	0.82	0.35	1.01
at 1000 cycles	0.02	N.T.	0.04	0.28	0.13	0.40	0.21	0.55
Dur-O-Wal Interior Tie								
at 30 cycles	N.T.	N.T.	N.T.	0.11	0.06	0.29	N.T.	0.23
at 1000 cycles	N.T.	N.T.	N.T.	0.11	0.06	0.29	N.T.	0.23
Rigid Datum - 1 flange connection								
at 30 cycles	N.T.	0.11	N.T.	0.25	N.T.	0.60	N.T.	0.52
at 1000 cycles	N.T.	0.11	N.T.	0.25	N.T.	0.60	N.T.	0.52
Rigid Datum - 2 flange connection								
at 30 cycles	N.T.	N.T.	N.T.	N.T.	N.T.	0.16	N.T.	0.13
at 1000 cycles	N.T.	N.T.	N.T.	N.T.	N.T.	0.16	N.T.	0.13

N.T. - not tested for cyclic displacement

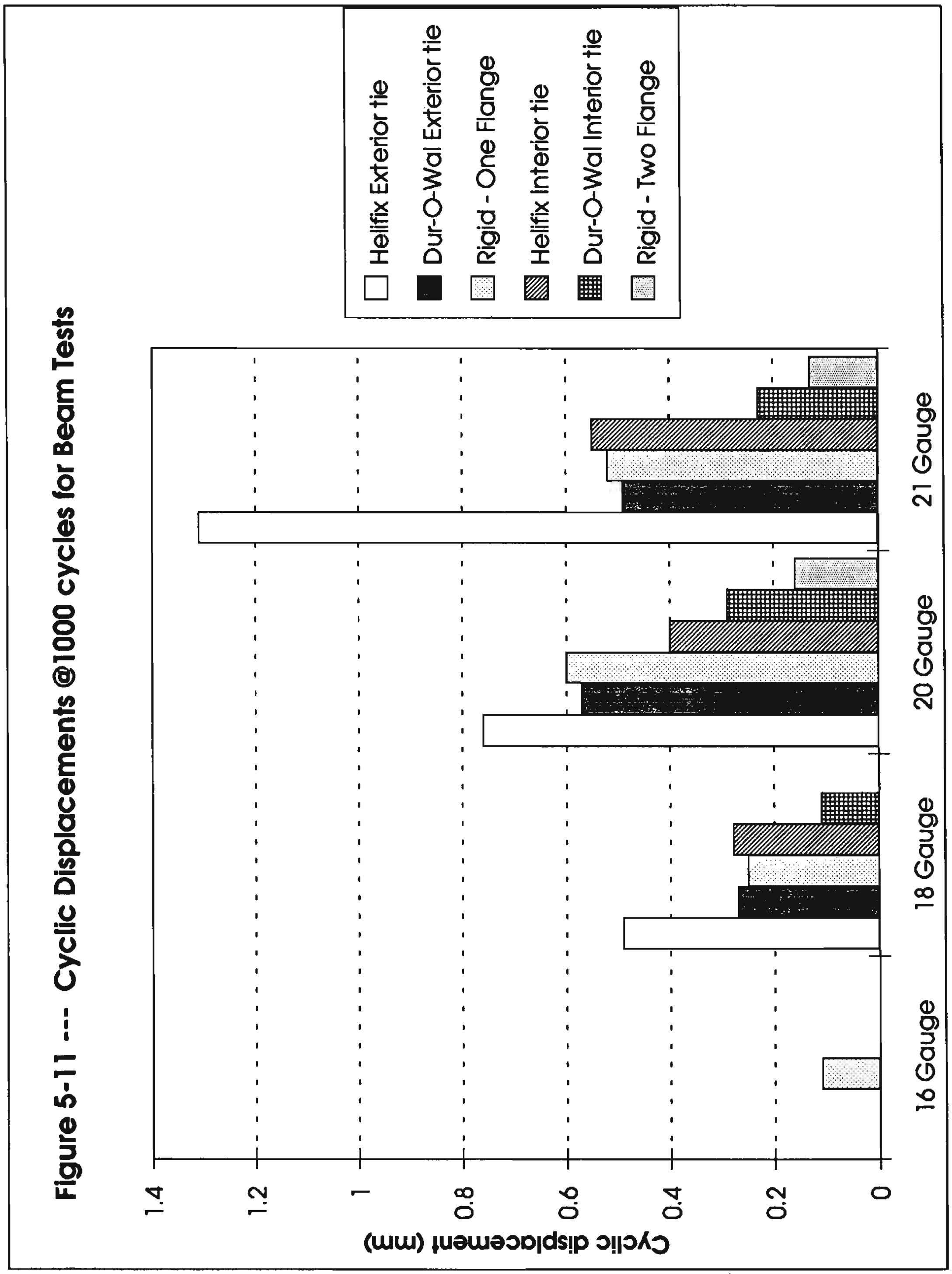


Figure 5-11 --- Cyclic Displacements @1000 cycles for Beam Tests

The CAN/CSA A370 code requires ties, including retrofit ties, to deform less than 2 mm when subjected to a load of 0.45 kN in tension or compression. To account for the variability within each test series characteristic values must be calculated as done with the maximum strengths. Table 5-9, 5-10, 5-11 and 5-12 contain the displacement at 0.45 kN values and the characteristic value for all test series conducted on the HE, DE, HI and DI ties respectively. The characteristic value is equal to the mean of the test series minus one and one half standard deviations. The extreme values, greater than 2mm that violate the code serviceability criterion, are shown in bold. As would be expected the displacement values for the beam tests are greater than the isolation tests.

From the values in Table 5-9 to 5-12 the governing characteristic displacement at 0.45 kN was determined for each tie and gauge. This governing characteristic value was determined by considering only the values from Test Series 2, 4, 5 and 6. The isolation tests in Test Series 1 and the beam tests with varying attachment location in Test Series 3 were not included. Table 5-13 lists the governing values. Figure 5-12 graphically illustrates these values. The Dur-O-Wal Exterior and Interior ties satisfy the serviceability requirement for all gauges. The Helifix Exterior and Interior tie meet the serviceability criteria only in the 16 gauge stud.

The attachment location has a significant influence on the characteristic displacement at 0.45 kN, particularly for exterior ties. Table 5-14 lists these values. Figure 5-13 illustrates the influence of the attachment location for all ties except the Dur-O-Wal Interior tie which was not tested with varying attachment location. The characteristic displacement at 0.45 kN doubles from the position closest to the web to the position out on the flange for the Helifix Exterior tie and increases by a factor of almost 10 for the Dur-O-Wal Exterior tie.

TABLE 5-9 -- Characteristic Displacement at 0.45 kN Values for Helifix Exterior Repair

Test Series	1 Average Displacement at 0.45 kN $\Delta_{0.45kN}$ mm	2 Standard Deviation $S.D.$ mm	3 Characteristic Displacement at 0.45 kN Δ_{char} mm	4 Rigid Datum Displacement at 0.45 kN Δ_{datum} mm	5 Corrected Characteristic Displacement at 0.45 kN Δ_{cor} mm
HE-1a-20	1.61	0.20	1.91	N.A.	N.A.
HE-1b-16	1.29	0.55	2.12	N.A.	N.A.
HE-1b-18	1.49	0.18	1.76	N.A.	N.A.
HE-1b-20	1.50	0.18	1.77	N.A.	N.A.
HE-1b-21	1.87	0.26	2.26	N.A.	N.A.
HE-1bA-20		0.00	0.00	N.A.	N.A.
HE-1c-16	1.09	0.50	1.84	N.A.	N.A.
HE-1c-18	1.50	0.26	1.89	N.A.	N.A.
HE-1c-20	1.54	0.38	2.11	N.A.	N.A.
HE-1c-21	1.68	0.16	1.92	N.A.	N.A.
HE-1d-20	1.49	0.21	1.81	N.A.	N.A.
HE-2-16	1.48	0.34	**1.99**	0.25	1.74
HE-2-18	2.24	0.22	2.57	0.55	2.02
HE-2-20	3.00	0.29	**3.44**	1.10	2.34
HE-2-21	3.84	0.84	**5.10**	1.60	3.50
HE-3-10	2.13	0.21	2.45	N.A.	N.A.
HE-3-20	3.00	0.29	3.44	1.10	2.34
HE-3-30	4.75	0.70	5.80	N.A.	N.A.
HE-4-16	1.29	0.42	1.92	N.A.	N.A.
HE-4-18	2.01	0.37	2.57	N.A.	N.A.
HE-4-20	2.98	0.28	3.40	1.30	2.10
HE-4-21	3.28	0.59	4.17	1.40	2.77
HE-4A-20		0.00	0.00	1.30	-1.30
HE-5-18	2.16	0.38	**2.73**	0.55	2.18
HE-5-20	2.68	0.43	3.33	1.10	2.23
HE-5-21	3.59	0.26	3.98	1.60	2.38
HE-6-20	2.74	0.26	3.13	1.30	1.83

$$\Delta_{char} = \Delta_{0.45kN} + 1.55 S.D.$$

$$\Delta_{cor} = \Delta_{char} - \Delta_{datum}$$

N.A. - not applicable as there were no rigid datum tests conducted for this test series

BOLD values are the highest or governing values based on Series 2, 4, 5 and 6

shaded cells indicate the standard deviation in which the coefficient of variation was greater than 20%

TABLE 5-10 -- Characteristic Displacement at 0.45 kN Values for Dur-O-Wal Exterior Repair

Test Series	Average Displacement at 0.45 kN $\Delta_{0.45kN}$ mm	Standard Deviation $S.D.$ mm	Characteristic Displacement at 0.45 kN Δ_{char} mm	Rigid Datum Displacement at 0.45 kN Δ_{datum} mm	Corrected Characteristic Displacement at 0.45 kN Δ_{cor} mm
DE-1a-20	0.31	0.06	0.40	N.A.	N.A.
DE-1b-20	0.44	0.05	0.52	N.A.	N.A.
DE-1c-16	0.13	0.01	0.15	N.A.	N.A.
DE-1c-18	0.21	0.01	0.23	N.A.	N.A.
DE-1c-20	0.39	0.06	0.48	N.A.	N.A.
DE-1c-21	0.88	0.11	1.05	N.A.	N.A.
DE-2-16	0.28	0.03	**0.33**	0.25	0.08
DE-2-18	0.53	0.04	0.59	0.55	0.04
DE-2-20	0.89	0.13	1.09	1.10	-0.02
DE-2-21	1.21	0.17	1.47	1.60	-0.14
DE-3-10	0.21	0.04	0.27	N.A.	N.A.
DE-3-20	0.89	0.13	1.09	1.10	-0.02
DE-3-30	1.79	0.19	2.08	N.A.	N.A.
DE-4-18	0.54	0.09	0.68	N.A.	N.A.
DE-4-20	1.24	**0.31**	**1.71**	1.30	0.41
DE-4-21	1.30	**0.48**	**2.02**	1.40	0.62
DE-5-18	0.54	**0.11**	**0.71**	0.55	0.16
DE-5-20	0.99	**0.21**	1.31	1.10	0.21
DE-5-21	1.23	0.15	1.46	1.60	-0.15
DE-6-20	1.15	0.07	1.26	1.30	-0.05

$$\Delta_{char} = \Delta_{0.45kN} + 1.55S.D.$$

$$\Delta_{cor} = \Delta_{char} - \Delta_{datum}$$

N.A. - not applicable as there were no rigid datum tests conducted for this test series

BOLD values are the highest or governing values based on Series 2, 4, 5 and 6

shaded cells indicate the standard deviation in which the coefficient of variation was greater than 20%

TABLE 5-11 -- Characteristic Displacement at 0.45 kN Values for Helifix Interior Repair

Test Series	Average Displacement at 0.45 kN $\Delta_{0.45kN}$ mm	Standard Deviation $S.D.$ mm	Characteristic Displacement at 0.45 kN Δ_{char} mm	Rigid Datum Displacement at 0.45 kN Δ_{datum} mm	Corrected Characteristic Displacement at 0.45 kN Δ_{cor} mm
HI-1a-20	1.66	0.83	2.91	N.A.	N.A.
HI-1b-16	0.77	0.46	1.46	N.A.	N.A.
HI-1b-18	1.45	0.22	1.78	N.A.	N.A.
HI-1b-20	1.63	0.32	2.11	N.A.	N.A.
HI-1b-21	2.18	0.97	3.64	N.A.	N.A.
HI-1c-16	1.07	0.44	1.73	N.A.	N.A.
HI-1c-18	1.45	0.53	2.25	N.A.	N.A.
HI-1c-20	1.76	0.43	2.41	N.A.	N.A.
HI-1c-21	2.79	0.61	3.71	N.A.	N.A.
HI-1d-20	1.34	0.54	2.15	N.A.	N.A.
HI-2-16	0.58	0.48	**1.30**	N.A.	N.A.
HI-2-18	1.62	0.26	2.01	N.A.	N.A.
HI-2-20	2.10	0.15	2.33	0.35	1.98
HI-2-21	2.63	0.61	**3.55**	0.42	3.13
HI-3-10	1.51	0.28	1.93	N.A.	N.A.
HI-3-20	2.10	0.15	2.33	0.35	1.98
HI-3-30	2.24	0.12	2.42	N.A.	N.A.
HI-4-16	0.37	0.09	0.51	N.A.	N.A.
HI-4-18	1.58	0.30	2.03	N.A.	N.A.
HI-4-20	2.41	0.69	**3.45**	N.A.	N.A.
HI-4-21	2.73	0.28	3.15	N.A.	N.A.
HI-5-18	1.77	0.24	**2.13**	N.A.	N.A.
HI-5-20	2.09	0.37	2.65	0.35	2.30
HI-5-21	2.83	0.46	3.52	0.42	3.10
HI-6-20	2.13	0.25	2.51	N.A.	N.A.

$$\Delta_{char} = \Delta_{0.45kN} + 1.55 S.D.$$

$$\Delta_{cor} = \Delta_{char} - \Delta_{datum}$$

N.A. - not applicable as there were no rigid datum tests conducted for this test series

BOLD values are the highest or governing values based on Series 2, 4, 5 and 6

shaded cells indicate the standard deviation in which the coefficient of variation was greater than 20%

TABLE 5-12 -- Characteristic Displacement at 0.45 kN Values for Dur-O-Wal Interior Repair

Test Series	Average Displacement at 0.45 kN $\Delta_{0.45kN}$ mm	Standard Deviation $S.D.$ mm	Characteristic Displacement at 0.45 kN Δ_{char} mm	Rigid Datum Displacement at 0.45 kN Δ_{datum} mm	Corrected Characteristic Displacement at 0.45 kN Δ_{cor} mm
DI-1b-16	0.11	0.05	0.19	N.A.	N.A.
DI-1b-18	0.20	0.07	0.31	N.A.	N.A.
DI-1b-20	0.27	0.06	0.36	N.A.	N.A.
DI-1b-21	0.28	0.04	0.34	N.A.	N.A.
DI-1d-20	0.31	0.13	0.51	N.A.	N.A.
DI-2-18	0.30	0.07	0.41	N.A.	N.A.
DI-2-20	0.35	0.03	0.40	0.35	N.A.
DI-2-21	0.51	0.23	**0.86**	0.42	N.A.
DI-4-16	0.17	0.01	**0.19**	N.A.	N.A.
DI-4-18	0.27	0.03	0.32	N.A.	N.A.
DI-4-20	0.62	0.27	1.03	N.A.	N.A.
DI-4-21	0.56	0.09	0.70	N.A.	N.A.
DI-6-18	0.48	0.23	0.83	N.A.	N.A.
DI-6-20	1.18	0.49	**1.92**	N.A.	N.A.
DI-6-21	0.61	0.12	0.79	N.A.	N.A.

$$\Delta_{char} = \Delta_{0.45kN} + 1.55 S.D.$$

$$\Delta_{cor} = \Delta_{char} - \Delta_{datum}$$

N.A. - not applicable as there were no rigid datum tests conducted for this test series
BOLD values are the highest or governing values based on Series 2, 4, 5 and 6
shaded cells indicate the standard deviation in which the coefficient of variation was greater than 20%

Table 5-13 -- Governing Characteristic Displacements at 0.45 kN

	Helifix Exterior Tie Δ_{char} mm	Dur-O-Wal Exterior Tie Δ_{char} mm	Helifix Interior Tie Δ_{char} mm	Dur-O-Wal Interior Tie Δ_{char} mm
16	1.99	0.33	1.30	0.19
18	2.73	0.71	2.13	0.83
20	3.44	1.71	3.45	1.92
21	5.10	2.02	3.55	0.86

Shaded cells indicate failure to meet serviceability criterion of maximum 2mm at 0.45 kN

Figure 5-12 --- Governing Characteristic Displacements at 0.45 kN

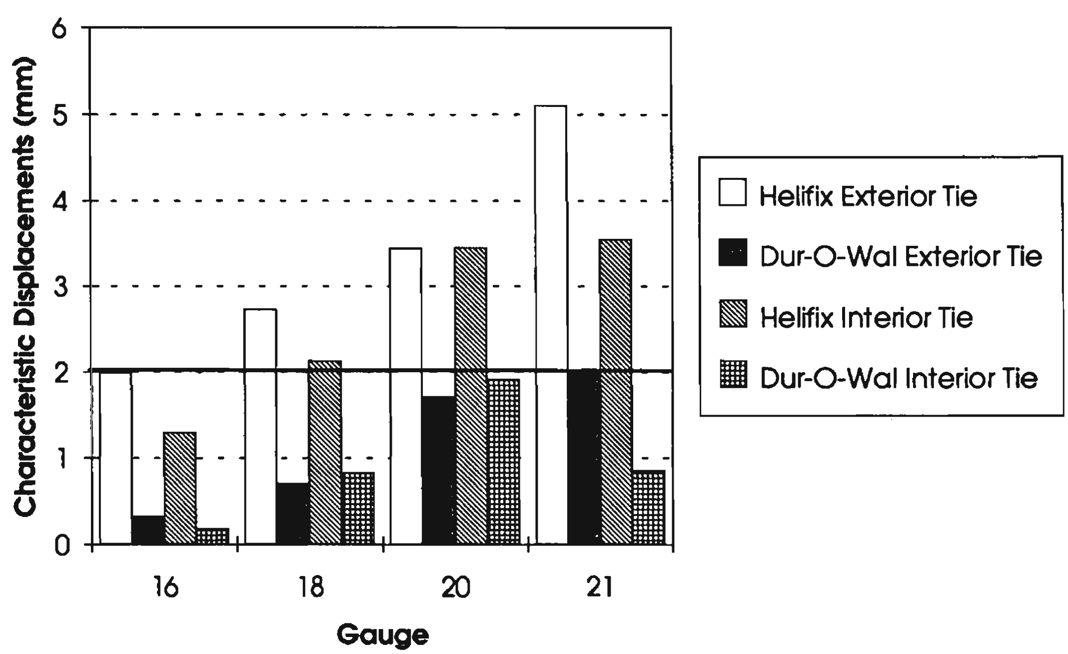


Table 5-14 -- Characteristic Displacements at 0.45 kN for Test Series 3

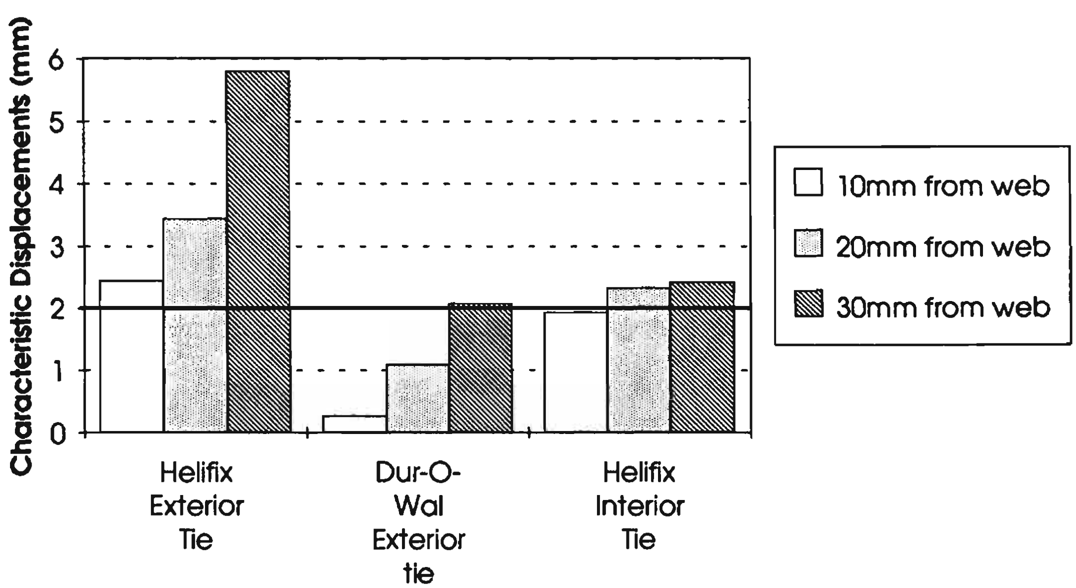
20 gauge studs

	Helifix Exterior Tie Δ_{char} mm	Dur-O-Wal Exterior Tie Δ_{char} mm	Helifix Interior Tie Δ_{char} mm	Dur-O-Wal Interior Tie Δ_{char} mm
10mm from web	2.445	0.27	1.93	N.T.
20mm from web	3.44	1.09	2.33	N.T.
30mm from web	5.80	2.08	2.42	N.T.

N.T. - no tests conducted for this test series

Shaded cells indicate failure to meet serviceability criterion of maximum 2mm at 0.45 kN

Figure 5-13 --- Characteristic Displacements at 0.45 kN for Test Series 3



The load versus displacement response is also informative. Figure 5-14 to Figure 5-17 show the characteristic load versus displacement curves for the first 4 mm of displacement for the exterior repairs in the four gauges of stud. In compression the two Dur-O-Wal ties follow the rigid one flange curve and exhibit greater stiffness than the two Helifix ties. In tension the Dur-O-Wal interior tie behaves similar to the rigid 2 flange connection. The Dur-O-Wal exterior tie is slightly stiffer initially than the rigid 1 flange connection. Both Helifix ties show a lesser degree of stiffness over the first 4 mm of displacement. Figures 5-18 to 5-21 show the characteristic load versus displacement curves for the first 4mm of displacement for the interior repairs in the four gauges of stud. The Helifix repairs show similar deformation response for each particular gauge, independent of whether the loading is in tension or compression. Although the Dur-O-Wal interior repair is less stiff in compression than in tension, the stiffness of these ties are greater than that for the Helifix ties.

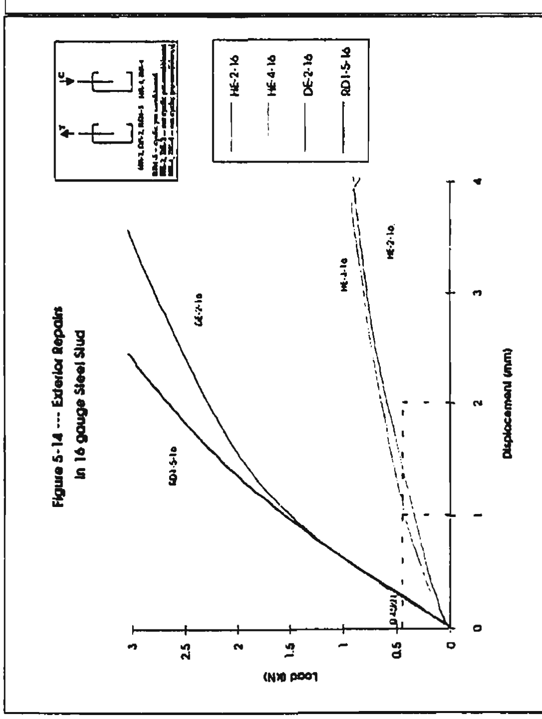


Figure 5-14 --- Exterior Repairs
In 16 gauge Steel Stud
Load (kN)
Displacement (mm)
HE-2-16
HE-4-16
DE-2-16
RD1-5-16

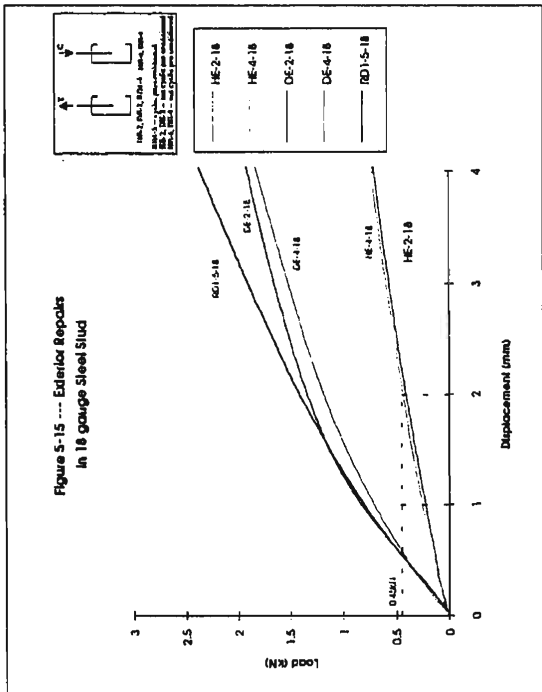


Figure 5-15 --- Exterior Repairs
In 18 gauge Steel Stud
Load (kN)
Displacement (mm)
HE-2-18
HE-4-18
DE-2-18
DE-4-18
RD1-5-18

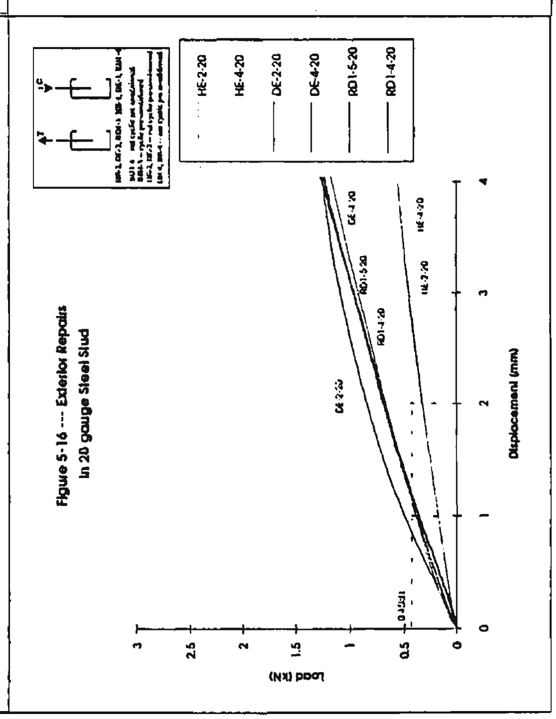


Figure 5-16 --- Exterior Repairs
In 20 gauge Steel Stud
Load (kN)
Displacement (mm)
HE-2-20
HE-4-20
DE-2-20
DE-4-20
RD1-5-20
RD1-4-20

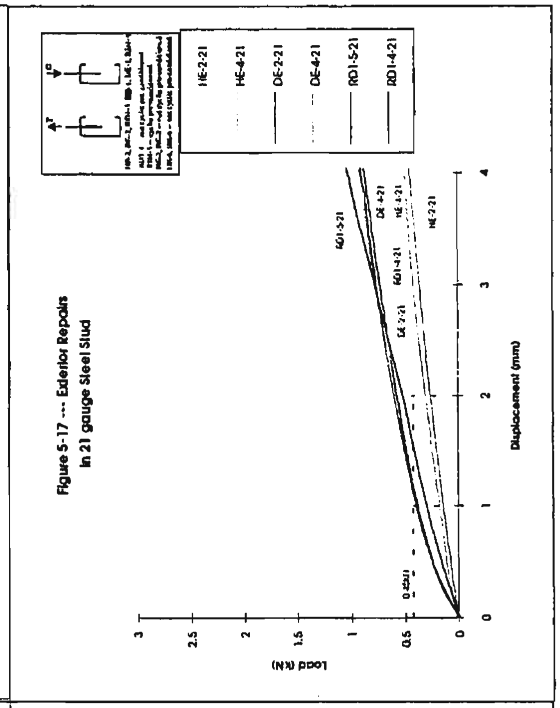


Figure 5-17 --- Exterior Repairs
In 21 gauge Steel Stud
Load (kN)
Displacement (mm)
HE-2-21
HE-4-21
DE-2-21
DE-4-21
RD1-5-21
RD1-4-21

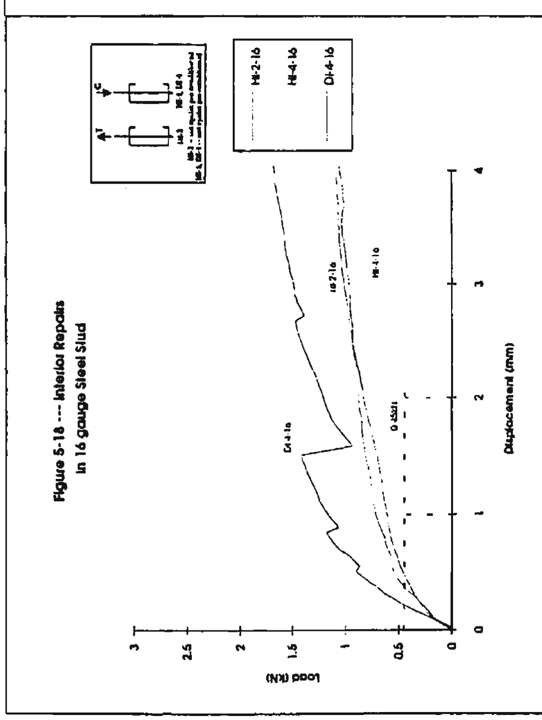


Figure 5-18 --- Interior Repairs in 16 gauge Steel Stud

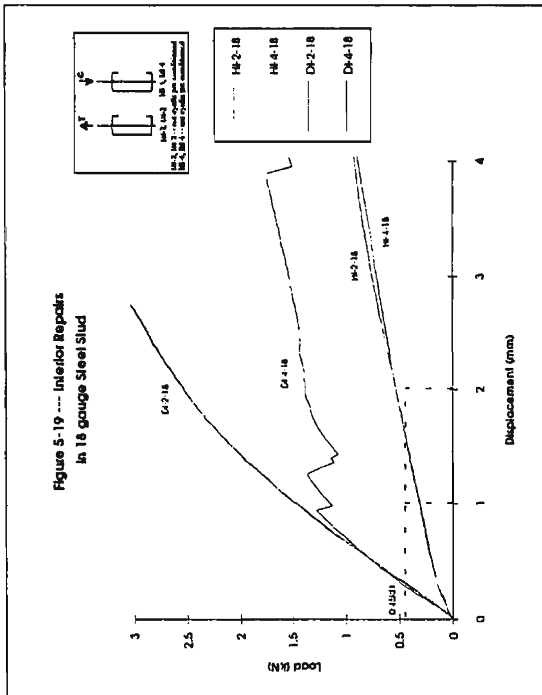


Figure 5-19 --- Interior Repairs in 18 gauge Steel Stud

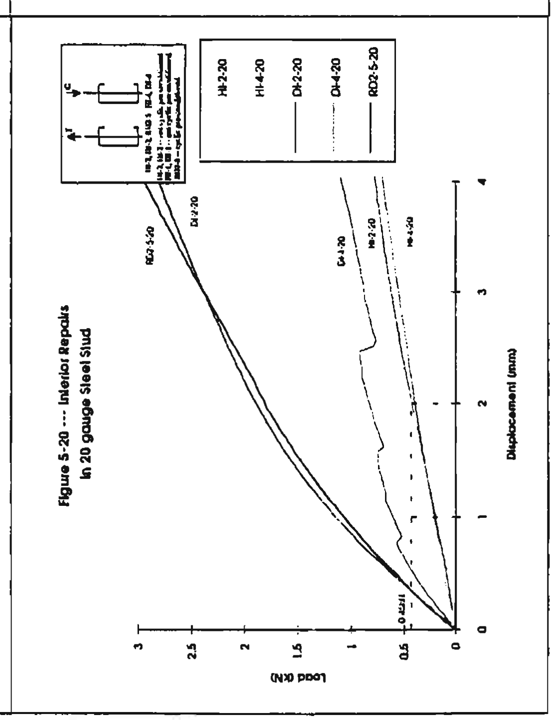


Figure 5-20 --- Interior Repairs in 20 gauge Steel Stud

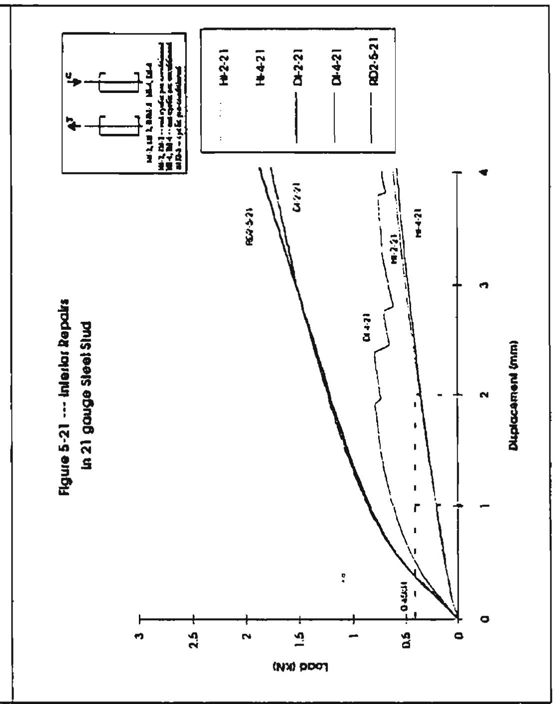


Figure 5-21 --- Interior Repairs in 21 gauge Steel Stud

5.3 Comparison of Retrofit Tie Results to McMaster Test Results

The McMaster study of tie systems for BV/SS wall systems was extensive but was directed at new construction. It is useful to compare the performance of the four retrofit ties to the new ties. The McMaster study dealt primarily with structural safety and serviceability. Only a written description of the nature of failure is found in the test reports, so direct comparison to the characteristic curves for the retrofit ties is not possible. The McMaster study found the Wrap-Around-Tie (WAT) to be clearly superior.

To compare on the basis of structural safety the averages and standard deviations from the tests at McMaster were tabulated along with the values for the retrofit ties; see Table 5-15.

The codes for the tie names have not been given here as the exact type of tie is not important. A detailed description of these new tie systems is contained in the McMaster study. Test Series 2 and 4 for the retrofit ties were chosen for comparison and represent tension and compression loading respectively. In the McMaster study the numeral 2 depicts tension and the numeral 1 depicts compression.

The standard deviation in the McMaster tests were, in general, smaller than for the retrofit ties even though both test programs used 5 tests as the sample size.

TABLE 5-15 -- Safety Comparisons of Retrofit Ties to Ties Used in New Construction

	Tie Name	1 Test Series	2 Average Maximum Load $\bar{x}$ kN	3 Standard Deviation $S.D.$ kN	4 Characteristic Strength R_{char} kN	5 Resistance for F.S. = 2.73 LSD R_w kN	6 Equivalent Number of Standard Deviations n F.S. = 2.73	7 1 in 10000 Resistance R_{10000} kN
Waterloo Retrofit Ties	HE	HE-2-20	1.65	0.16	1.410	0.516	7.08	1.042
		HE-4-20	0.88	0.1	0.730	0.267	6.13	0.500
	HI	HI-2-20	1.1	0.05	1.025	0.375	14.49	0.910
		HI-4-20	1.65	0.19	1.365	0.500	6.05	0.928
	DI	DI-2-20	4.45	0.33	3.955	1.449	9.09	3.196
		DI-4-20	2.33	0.46	1.640	0.601	3.76	0.582
	DE	DE-2-20	2.17	0.25	1.795	0.658	6.05	1.220
		DE-4-20	2.42	0.12	2.240	0.821	13.33	1.964
McMaster Ties for New Construction	DLA	T-1-2	1.709	0.186	1.430	0.524	6.37	1.002
		T-1-1	1.678	0.136	1.474	0.540	8.37	1.161
	CTA	T-2-2	2.191	0.039	2.133	0.781	36.15	2.043
		T-2-1	2.278	0.043	2.214	0.811	34.12	2.115
	SSA	T-3-2	1.168	0.042	1.105	0.405	18.17	1.008
		T-3-1	•	•	•	•	•	•
	SLA	T-4-2	1.004	0.07	0.899	0.329	9.64	0.738
		T-4-1	1.088	0.071	0.982	0.360	10.26	0.818
	WLA	T-5-2	1.288	0.059	1.200	0.439	14.38	1.064
		T-5-1	2.458	0.034	2.407	0.882	46.36	2.329
	FLA	T-6-2	1.129	0.069	1.026	0.376	10.92	0.867
		T-6-1	1.761	0.069	1.658	0.607	16.72	1.499
	WAT	T-7-2	2.642	0.122	2.459	0.901	14.27	2.178
		T-7-1	4.5	0.09	4.365	1.599	32.23	4.158
	SDT	T-8-2	1.418	0.044	1.352	0.495	20.97	1.251
		T-8-1	•	•	•	•	•	•
	DW10	T-9-2	2.007	0.068	1.905	0.698	19.25	1.749
		T-9-1	•	•	•	•	•	•
	CST	T-10-2	1.481	0.044	1.415	0.518	21.88	1.314
		T-10-1	0.699	0.044	0.633	0.232	10.62	0.532
	TA	T-12-2	4.101	0.444	3.435	1.258	6.40	2.414
		T-12-1	3.514	0.482	2.791	1.022	5.17	1.682

Notes :
1) all tests conducted on 20 gauge studs
2) Code of tests
This Study
Series ?-2-20 -- tension
Series ?-4-20 -- compression
McMaster Study
Series T-?-2 -- tension
Series T-?-1 -- compression

Figure 5-22 illustrates the characteristic strength for all the tie types in both tension and compression for 20 gauge studs. Figure 5-23 illustrates the governing resistance values, using a factor of safety of 2.73, for all tie types in 20 gauge studs. The strength of the retrofit ties is greater or comparable to many of the new ties. Some of these ties are not used anymore due to their low stiffness. The strength of many of these ties were limited by the pullout of the screws. For ties that engage the web, like TA and WAT, or in the retrofit case engage both flanges, DI, the strengths are typically higher than the ties that rely on the connection to the exterior flange of the stud cross-section.

Although characteristic curves for the entire tests are not provided in the McMaster test report, regression coefficients are given for each test series. Using the regression coefficients a comparison of the initial pullout response of the new and retrofit ties is possible. Figures 5-24 and 5-25 illustrate the characteristic tie load versus deflection curves for all the ties in 20 gauge steel stud for tension and compression respectively. The curves are limited to 4 mm as the McMaster study deemed this the performance domain and did not consider any response after 4mm to be important. In comparing the characteristic curves in this way the difference in the two setups should be restated. The McMaster study had a span of 450mm while this study used a span of 400mm. This factor will give the retrofit ties a slightly stiffer response. The other important difference is that the McMaster study used steel studs that had a 31mm flange as opposed to 41mm in this test program. The significant effect of the attachment location on the stiffness of the connection, particularly for exterior ties has been demonstrated in Figure 5-13. The smaller flange for the tests on new ties will give them a more stiff initial response.

From Figure 5-24 it is apparent that many ties perform initially as a rigid one flange connection implying that no slip of the tie in the flange has occurred. The ties that are less stiff than the rigid one flange connection likely have slipped in the flange at the screw points.

The Dur-O-Wal Interior tie is the stiffest tie in tension for 20 gauge while it has an average stiffness in compression. The Helifix ties are shown to be less stiff than many of the new ties. The Dur-O-Wal Exterior tie exhibited a comparable amount of stiffness to many of the new tie systems.

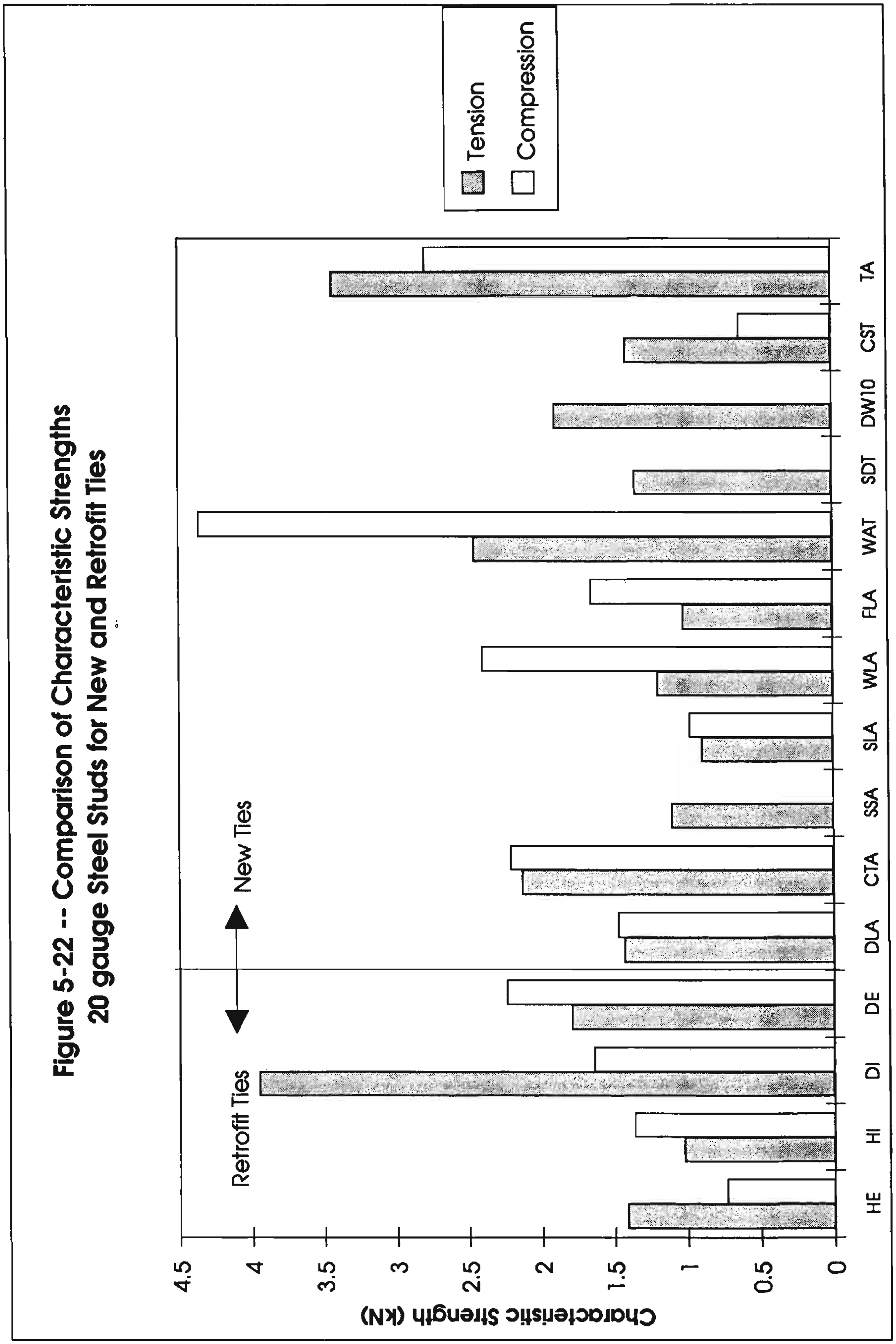


Figure 5-22 -- Comparison of Characteristic Strengths 20 gauge Steel Studs for New and Retrofit Ties

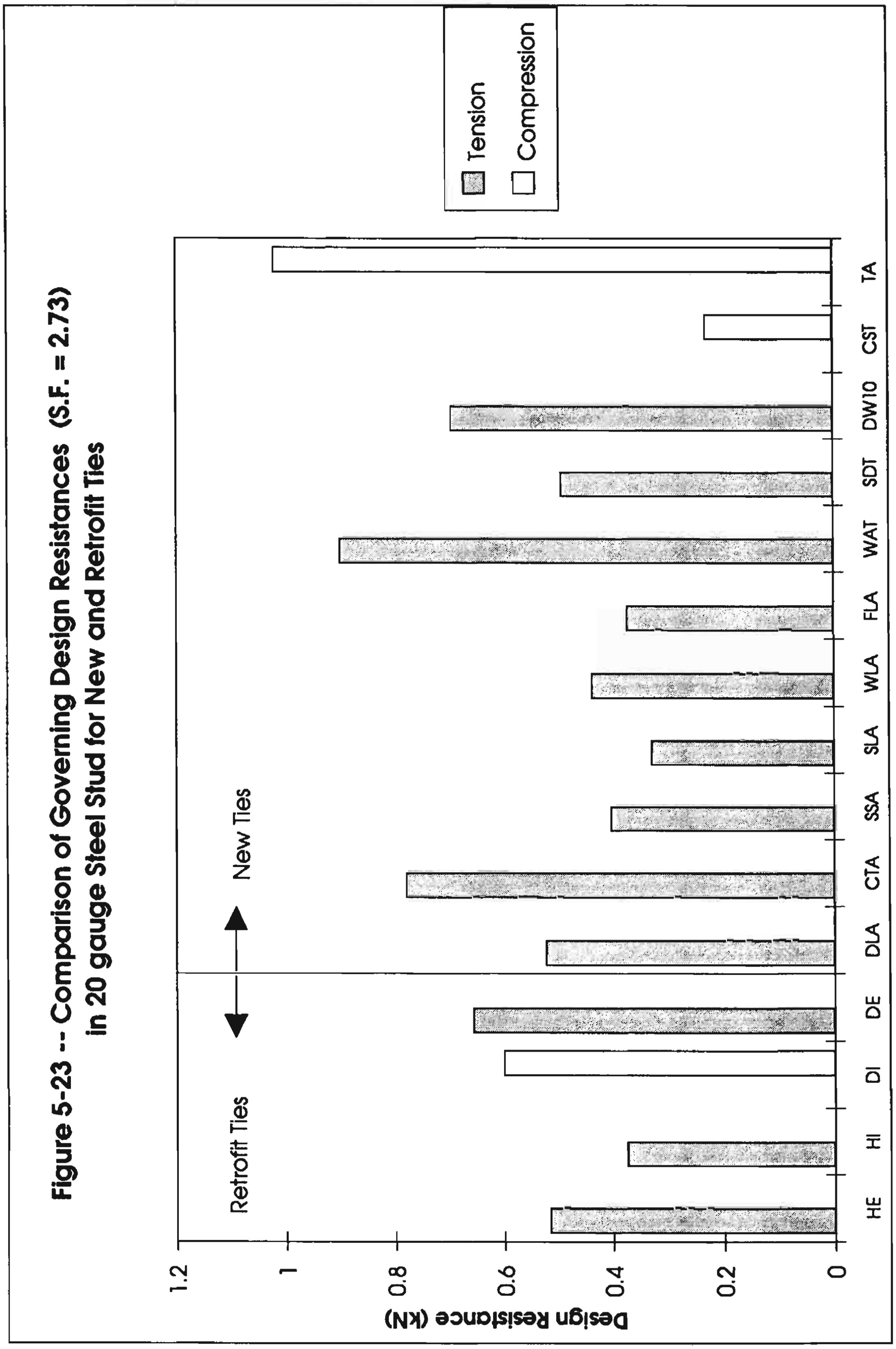
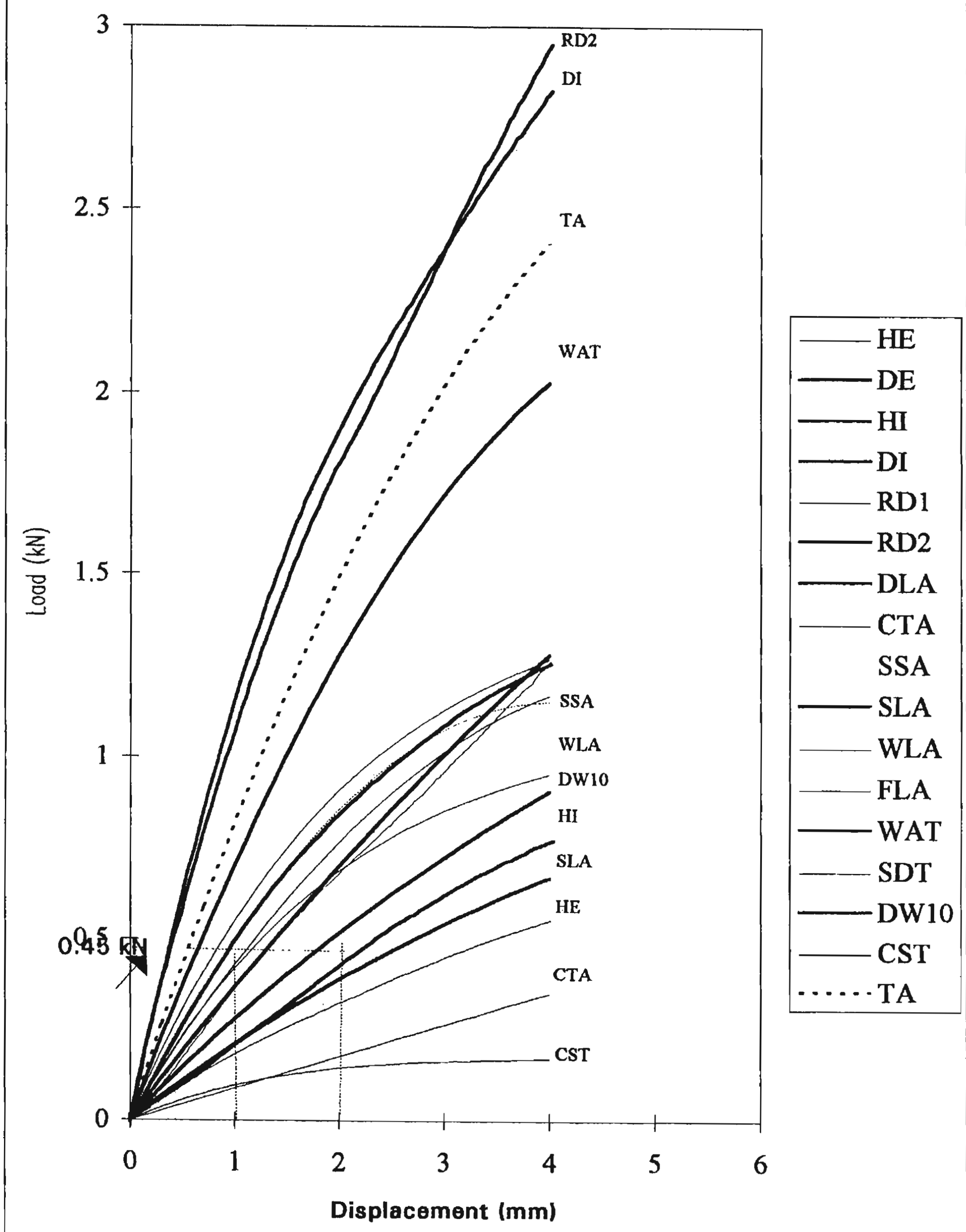


Figure 5-23 -- Comparison of Governing Design Resistances (S.F. = 2.73) in 20 gauge Steel Stud for New and Retrofit Ties

Figure 5-24 -- Characteristic Curves for Tension 20 Gauge



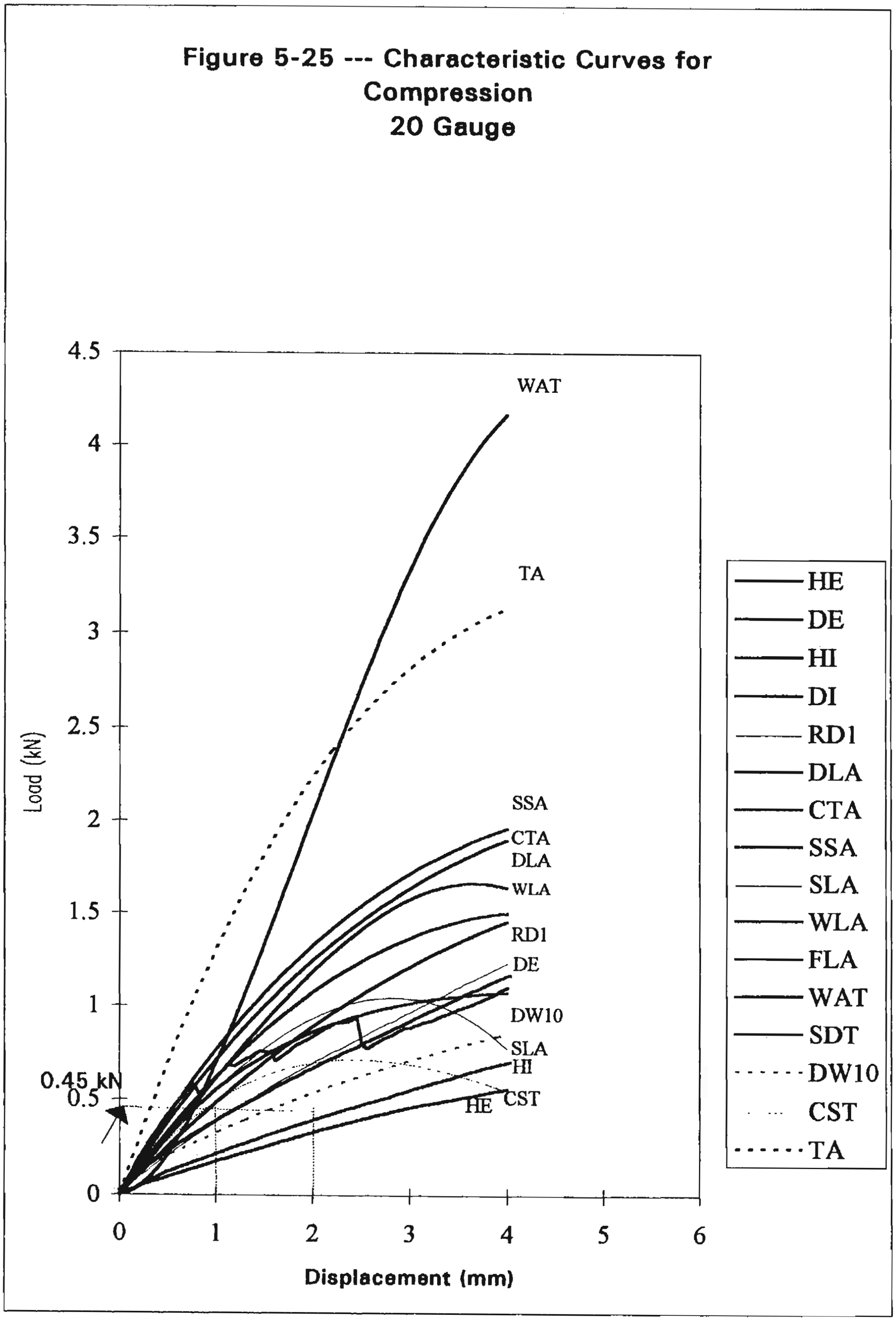


Figure 5-25 --- Characteristic Curves for Compression
20 Gauge
Load (kN)
Displacement (mm)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
0
1
2
3
4
5
6
0.45 kN
WAT
TA
SSA
CTA
DLA
WLA
RD1
DE
DW10
SLA
HI
HE
CST
HE
DE
HI
DI
RD1
DLA
CTA
SSA
SLA
WLA
FLA
WAT
SDT
DW10
CST
TA

The serviceability requirement considered in the McMaster study was a deflection limit of 1.2mm at 0.45 kN as recommended by the Brick Institute of America. Test values were not given for the displacements at 0.45kN. Table 5-16 contains the load at 1 and 2 mm for the retrofit ties for Test Series 2 and 4, tension and compression respectively. The load at 1.2 mm for the new ties from the McMaster report are also listed. Equivalent stiffness values for the retrofit ties have been calculated. Interpolation of the stiffness values for the retrofit ties gives values that can be directly compared to the stiffness for the new ties. Figure 5-26 illustrates these stiffness values. The Helifix ties have a low stiffness when compared to most of the other ties. The Dur-O-Wal repair ties have comparable stiffness values.

TABLE 5-16 -- Serviceability Comparisons of Retrofit Ties to Ties Used in New Construction

	Tie Name	Test Series	Load at 1 mm kN	Load at 1.2 mm kN	Load at 2 mm kN	**Stiffness** Load divided by 1.0 mm N/mm	**Stiffness** Load divided by 1.2 mm N/mm	**Stiffness** Load divided by 2.0 mm N/mm
Waterloo Retrofit Ties	HE	HE-2-20	0.180		0.310	180	172	155
		HE-4-20	0.190		0.360	190	187	180
	HI	HI-2-20	0.210		0.420	210	210	210
		HI-4-20	0.220		0.420	220	217	210
	DI	DI-2-20	1.140		1.900	1140	1077	950
		DI-4-20	0.670		0.880	670	593	440
	DE	DE-2-20	0.500		0.840	500	473	420
		DE-4-20	0.390		0.660	390	370	330
McMaster Ties for McMaster New Construction	DLA	T-1-2		0.436			363	
		T-1-1		0.730			608	
	CTA	T-2-2		0.104			87	
		T-2-1		0.826			688	
	SSA	T-3-2		0.582			485	
		T-3-1		0.888			740	
	SLA	T-4-2		0.245			204	
		T-4-1		0.644			537	
	WLA	T-5-2		0.484			403	
		T-5-1		0.718			598	
	FLA	T-6-2		0.505			421	
		T-6-1		0.570			475	
	WAT	T-7-2		0.829			691	
		T-7-1		0.930			775	
	SDT	T-8-2		0.635			529	
		T-8-1		0.626			522	
	DW10	T-9-2		0.328			273	
		T-9-1		0.372			310	
	CST	T-10-2		0.108			90	
		T-10-1		0.551			459	
	TA	T-12-2		1.055			879	
		T-12-1		1.515			1263	

Notes
1) all tests conducted on 20 gauge studs
2) Code of tests
   Series ?-20-2 - tension
   Series T-?-2 - tension
   Series ?-20-4 - compression
   Series T-?-1 - compression
3) Shaded Cells are based on interpolated Load at 1.2 mm values

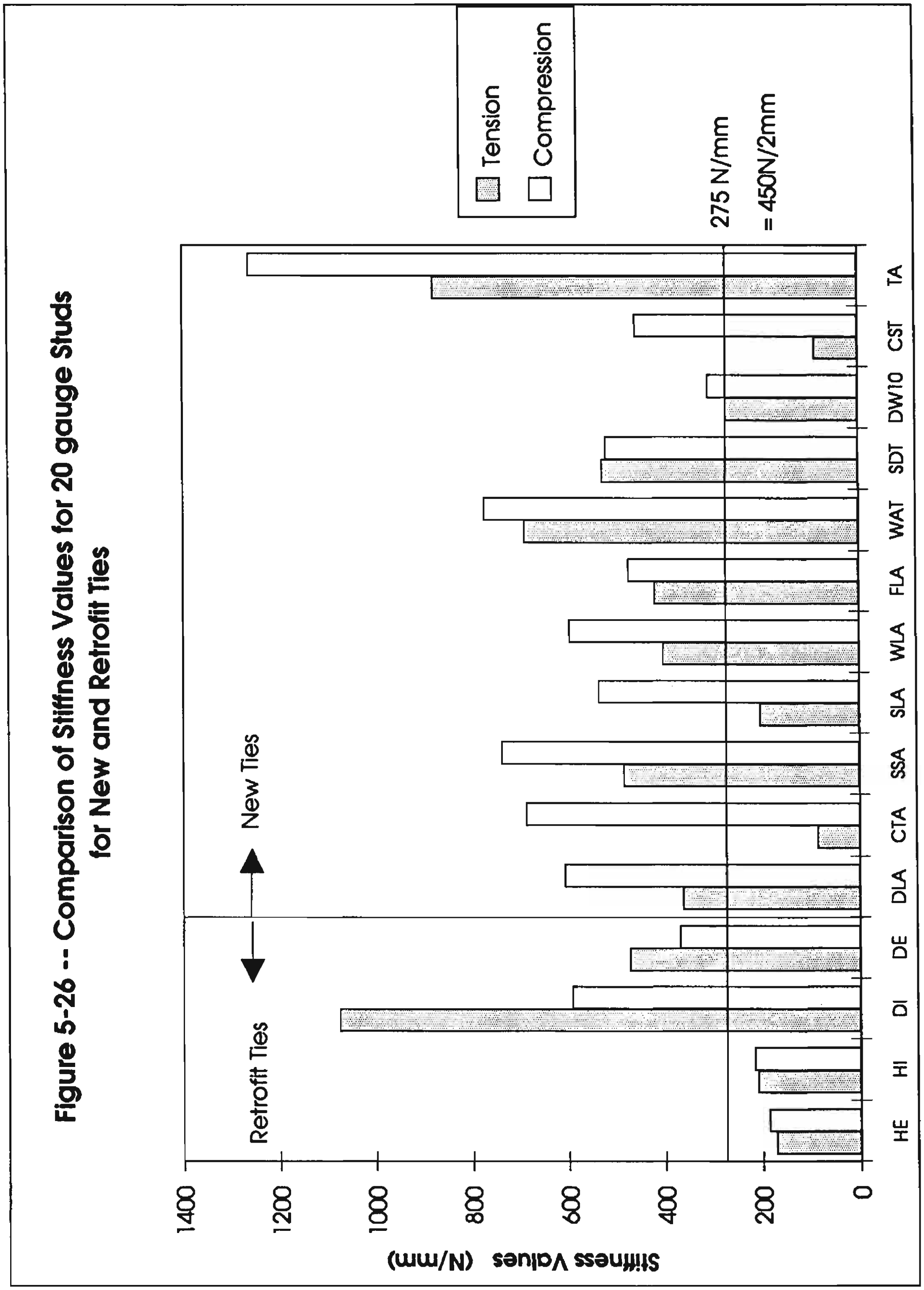


Figure 5-26 -- Comparison of Stiffness Values for 20 gauge Studs for New and Retrofit Ties

6. CONCLUSIONS

6.1 Design Recommendations

Four proprietary retrofit tie systems have been tested with regard to the nature of the tie to steel stud connection. Their performance has been assessed with respect to various performance requirements. This evaluation has largely been restricted to structural performance, specifically structural safety and serviceability. Other performance considerations such as the effect on wall system air leakage, corrosion potential, thermal bridging, etc. are the subject of a separate study (Task 3). In an earlier study (Task 1) the technical and economic feasibility of eleven retrofit tie systems were assessed. For Task 2, four tie systems, two interior and two exterior fixes, were chosen from the eleven in Task 1, for developmental and conformance testing.

Of the four retrofit systems tested it has been established that only one system, the Dur-O-Wal Exterior fix (using a Dur-O-Wal 1/4" dia. lagbolt into the steel stud and an expansion anchor to the brick) consistently developed sufficient strength, with satisfactory initial stiffness and acceptable displacements, to be used with all the gauges of steel stud framing tested, i.e., 16, 18, 20, and 21 gauge. A summary of pertinent characteristics, suitable for use by a designer, is presented in Table 6-1. Note that the gauge of the vertical stud is an important parameter. While 22, 24 or even thinner gauge studs have been used in many buildings, one should be very careful in any attempt to extrapolate the Task 2 test results to these thin steel stud sections.

The pertinent design characteristics for the other 3 tie systems are also presented in a similar format. The reservations, comments, etc. in Tables 6-2, 6-3, and 6-4 should be noted. In particular it should be emphasized that :-

- **Dur-O-Wal Exterior Fix** (lagbolt and expansion anchor) - This tie system may be used with 16, 18, 20 and 21 gauge studs, but special care should be exercised with the lighter gauge steel stud framing.

- **Helifix Exterior Fix** (HRT80, dry fix in SS, polyester resin in BV) - This retrofit fix is not recommended except, perhaps for use with 16 or thicker gauge steel. This tie connection is the least stiff of the four systems tested.

- **Helifix Interior Fix** (HRT80, Tie Dry Fixed) - This retrofit tie system is suitable for use with 16 and, perhaps, 18 gauge steel stud.

- **Dur-O-Wal Interior Fix** (Stainless Steel Rod and Sleeve with Epoxy) - This tie system is suitable for use with all the tested gauges of steel stud but the tie capacity is, in all cases, relatively low.

These recommendations can be summarized in a single chart, Table 6-5, which provides a simple means for choosing the appropriate supplementary tie system. These findings essentially meet the first two stated objectives for Task 2 i.e.:

(i) to evaluate and assess the capabilities of four retrofit tie systems; two suited to retrofit from the exterior and two for installation working solely from the interior

(ii) to identify and apply the relevant performance requirements to the four retrofit tie systems.

The third objective, to discuss various issues, has largely been dealt with in the text, in particular Chapter 2 and 5, but additional comment is required on the following three issues.

Table 6-1 - Design Values for Dur-O-Wal Exterior Tie
Type of Tie : Dur-O-Wal Exterior Tie

Structural Steel Stud Framing: Gauge	Structural Steel Stud Framing: Drill Hole Diameter ϕ inch	SERVICE LOAD CAPABILITIES: Design Strength at Service Load R_w kN 1	Characteristic Displacement at 0.45 kN: Δ_{beam} mm 2	Characteristic Displacement at 0.45 kN: Δ_{tie} mm 3	Cyclic Displacement +/- 0.15 kN 1000 cycles Δ mm 4	Initial Stiffness $\frac{R}{\Delta}$ N/mm 5	COMMENTS
16	3/16"	1.3	0.33	0.08	0.03	1500	
18	5/32"	0.77	0.71	0.16	0.27	800	
20	5/32"	0.6	1.71	0.41	0.57	400	
21	5/32"	0.43	2.02	0.62	0.49	350	Marginal

Notes :
1) This should only be used in a retrofit situation; 20 and 21 gauge steel framing are not recommended for new construction
2) Based on the measured load in a beam test at an overall displacement of 2mm this displacement is primarily displacement of the steel stud.
The tie does not contribute in any significant way to this displacement.
3) Based on the measured load in a beam test at an overall displacement of 0.45 kN.
4) Based on a beam test and 1000 cycles of +0.15 kN (33 Lbf) to -0.15 kN (33 Lbf) loading
5) Based on the measured load in a beam test at an overall displacement of 0.45 kN

Table 6-2 - Design Values for Helifix Exterior Tie
Type of Tie : Helifix Exterior Tie

Structural Steel Stud Framing: Gauge	Structural Steel Stud Framing: Drill Hole Diameter ϕ inch	SERVICE LOAD CAPABILITIES: Design Strength at Service Load R_w kN [1]	Characteristic Displacement at 0.45 kN: Δ_{beam} mm [2]	Characteristic Displacement at 0.45 kN: Δ_{tie} [3]	Cyclic Displacement +/- 0.15 kN 1000 cycles Δ mm [4]	Initial Stiffness $\frac{R}{\Delta}$ N/mm [5]	COMMENTS
16	1/4"	0.50	1.99	1.74	0.69	350	
18	1/8"	0.40	2.73	2.18	0.76	200	Not Recommended
20	3/32"	-	-	-	-	-	Not Recommended
21	3/32"	-	-	-	-	-	Not Recommended

Notes :
1) Values are for a retrofit situation; 20 and 21 gauge steel framing are not recommended for new construction
2) Based on the beam test
3) Tie displacement = Beam displacement - displacement for Rigid Datum (i.e. beam only)
4) Based on a beam test and 1000 cycles of +0.15 kN (33 Lbf) to -0.15 kN (33 Lbf) loading
5) Based on the measured load in a beam test at an overall displacement of 0.45 kN.

Table 6-3 - Design Values for Helifix Interior Tie

Type of Tie : Helifix Interior Tie

Structural Steel Stud Framing: Gauge	Structural Steel Stud Framing: Drill Hole Diameter ϕ inch	SERVICE LOAD CAPABILITIES: Design Strength at Service Load R_w kN 1	Characteristic Displacement at 0.45 kN: Δ_{beam} mm 2	Characteristic Displacement at 0.45 kN: Δ_{tie} 3	Cyclic Displacement +/- 0.15 kN 1000 cycles Δ mm 4	Initial Stiffness $\frac{R}{\Delta}$ N/mm5	COMMENTS
16	1/4"	0.50	1.73	-	-	500	
18	1/4"	0.40	2.13	-	0.41	250	Marginal
20							Not Recommended
21							Not Recommended

Notes :

1) Values are for a retrofit situation; 20 and 21 gauge steel framing are not recommended for new construction
2) Based on the beam test
3) Tie displacement = Beam displacement - displacement for Rigid Datum (i.e. beam only)
4) Based on a beam test and 1000 cycles of +0.15 kN (33 Lbf) to -0.15 kN (33 Lbf) loading
5) Based on the measured load in a beam test at an overall displacement of 0.45 kN.

Table 6-4 - Design Values for Dur-O-Wal Interior Tie
Type of Tie : Dur-O-Wal Interior Tie

Structural Steel Stud Framing		SERVICE LOAD CAPABILITIES					
Gauge	Drill Hole Diameter ϕ inch	Design Strength at Service Load R_w kN 1	Characteristic Displacement at 0.45 kN Δ_{beam} mm 2	Characteristic Displacement at 0.45 kN Δ_{tie} 3	Cyclic Displacement +/- 0.15 kN 1000 cycles Δ mm 4	Initial Stiffness $\frac{R}{\Delta}$ N/mm5	COMMENTS
16	3/8"	0.45	0.19	-	-	2000	
18	3/8"	0.45	0.83	-	0.11	1200	
20	3/8"	0.45		-	0.29	800	
21	3/8"	0.36	0.86	-	0.23	700	

Notes :
1) Values are for a retrofit situation; 20 and 21 gauge steel framing are not recommended for new construction
2) Based on the beam test
3) Tie displacement = Beam displacement - displacement for Rigid Datum (i.e. beam only)
4) Based on a beam test and 1000 cycles of +0.15 kN (33 Lbf) to -0.15 kN (33 Lbf) loading
5) Based on the measured load in a beam test at an overall displacement of 0.45 kN.

Table 6-5 - Design Service Loads for Retrofit Tie Systems
(Tie to Stud Connection)

Steel Stud Gauge	Exterior Retrofit		Interior Retrofit		
	Helifix	Dur-O-Wal	Helifix	Dur-O-Wal	
16	0.50	1.30	0.50	0.45	Suitable for both retrofit and new construction
18		0.77	0.40	0.45	
20		0.60		0.45	Suitable for retrofit only
21		0.43		0.36	

Marginal
Not Suitable

6.2 Wall Design

Evaluating the wind load on a tie within a BV/SS wall system involves calculating the wind pressure at the appropriate height for some specific location. The product of the wind pressure and the tributary area usually establishes the required capacity of the tie. As discussed in Section 2, the following expression is generally used to evaluate the relevant wind pressure :

$$p = qC_pC_gC_e$$

The load on the tie is thus given by:

$$R = Ap = AqC_pC_gC_e$$

The factor, C_p , is the pressure coefficient; in the worst case, high local suction on walls at the corners of the building, has a value of -1.0 (negative meaning suction). For cladding a value of 2.5 is used for the gust factor, C_g . It follows that:

$$R = -Aq2.5C_e$$

The exposure factor, C_e, for the purposes of evaluating tie loads, is best estimated by the one fifth power law expression, as follows with the height of the tie given by h:

$$C_e = (\mathrm{h}/10)^{0.2}$$

This results in the following expression for the tie load:

$$R = -Aq2.5\ (\mathrm{h}/10)^{0.2}$$

This expression can be reduced to:

$$R = -1.577\,Aq\ (h)^{0.2}$$

The stagnation wind pressure value, q, is found from tables in the Supplement to the NBCC. For ties this reference pressure is based on a wind with a 10 year recurrance interval measured at a height of 10 metres above grade for the appropriate location.

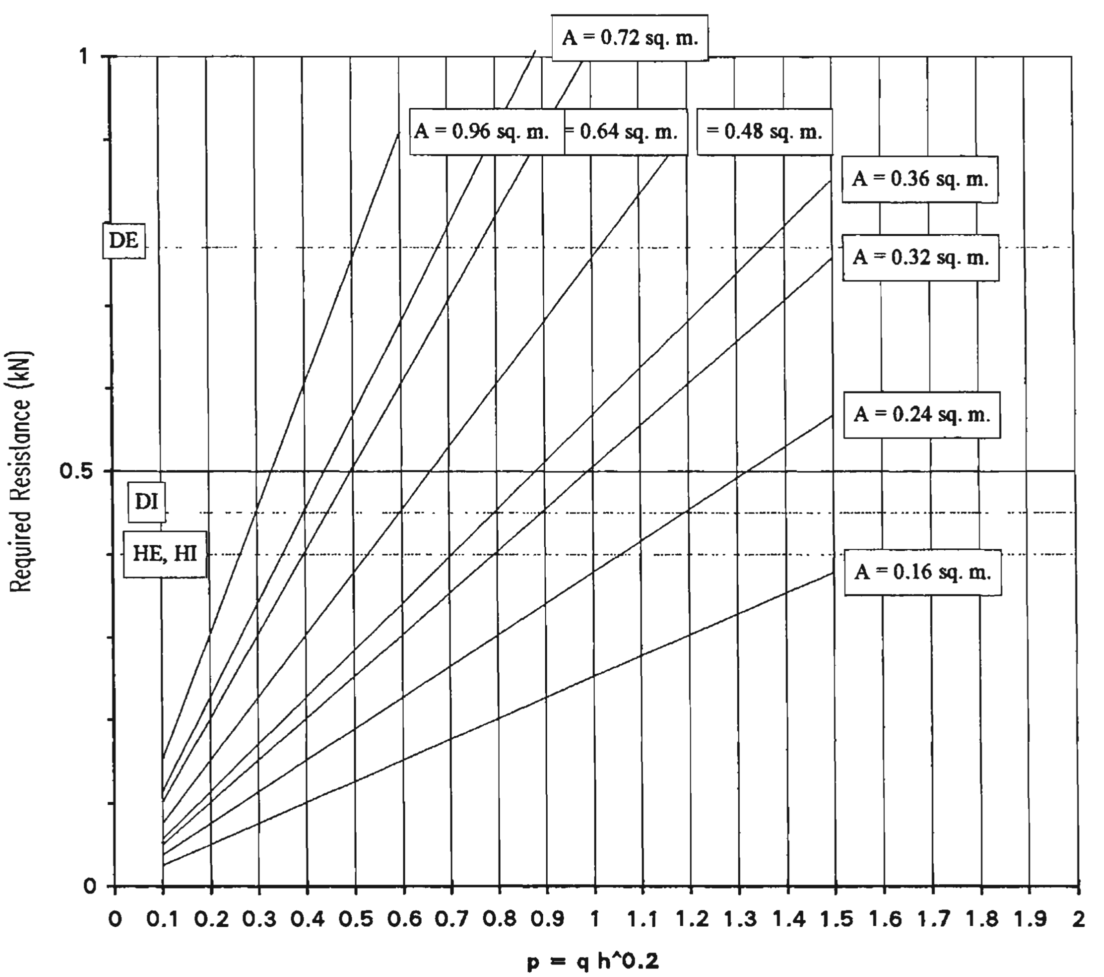
To demonstrate the practical relevance of the previous design recommendations for the tie-SS connection for walls on buildings of different height, in different locations and different tributary areas for the ties, Figure 6-1 has been provided. The horizontal axis represents the lateral wind pressure , p, given in terms of the reference velocity pressure for the wind, q, and the height of the tie involved, z. On the vertical axis the required working load (or service level or the unfactored load) on the tie, R_w is given for various "equivalent" tributary areas. The term "equivalent" tributary areas is used in order to ensure that consideration is given to the structural interaction between the BV and the SS. This is particularly important if the wall system has what may be considered to be a flexible backup. With a flexible backup it has been recommended that the equivalent tributary area may be calculated as either :

(i) twice the actual tributary area, or
(ii) 0.4 times the tributary area on the stud strip for each floor height

R_w values for the four retrofit tie systems connected to, for example, 18 gauge steel stud framing have also been superimposed on Table 6-6.

To apply Figure 6.1 to a particular building, the value for the reference velocity pressure value, q, must first be selected from the tables in the Supplement to the NBCC. Using the Table at the base of Figure 6.1, the appropriate value for p can then be determined. The appropriate retrofit tie system can then be selected directly from the design chart.

Figure 6-1 --- Required Resistances for Equivalent Tributary Areas (18 gauge Steel Stud)



LOCATION	q	qh^0.2 values for height (metres)				
	kPa	10	20	30	50	100
Montreal, Ottawa, Fredriction	0.3	0.48	0.55	0.59	0.66	0.75
Toronto, Halifax, Calgary	0.4	0.63	0.73	0.79	0.87	1.00
Vancouver	0.45	0.71	0.82	0.89	0.98	1.13

6.3 Exterior or Interior Retrofit

For reasons that really have nothing to do with tie performance, a very important decision is whether to work from the inside or outside when providing supplementary ties. If access is available, if the noise and dust can be tolerated, and if the damage can be limited so that tear down, fix-up and finish costs are kept reasonable, then there are very real advantages to working from the interior. For example staging can be avoided and, to some extent, weather can be eliminated as a factor. In many, if not most, buildings, for social and other non-technical reasons, the work will probably have to be done from the exterior.

Largely because the interior fixes that were tested had to penetrate both flanges, the interior fixes tend to behave rather differently; they were stiffer and potentially stronger than a comparable exterior fix. In general an interior fix is likely to be stiffer than an exterior fix because of the attachment of the tie to both flanges. For instance the Helifix Interior tie had less cyclic load displacement, less displacement at 0.45 kN and higher initial stiffness than the Helifix Exterior Tie. If the interior sheathing (gypsum board) is well connected to the stud the amount of flange rotation will also be significantly reduced.

However, the Dur-O-Wal Interior tie did not perform better than the Exterior Dur-O-Wal tie; but it must be noted that very different tie systems were used. This is one reason why categorizing exterior or interior repairs as good one relative to the other, can be misleading especially when the same supplier distributes two or more different tie systems.

There are some other aspects to interior and exterior repair work that are important. An interior tie requires sealing and patching over with drywall. If relative movement, even slight movement, of the tie were to occur visible damage to the interior face of the drywall would be visible from the inside.

6.4 Comparison of Test Setup to Real Wall Conditions

Physically modeling the actual behaviour of a steel stud wall in a test setup is, in fact, rather difficult. The test setup should simulate the displacements that could occur in a wall. Tie spacing and location, support conditions, wall height, presence and type of exterior sheathing, influence of interior sheathing, etc. all affect how the wall will perform.

The test setup should ideally

- enable initial stiffnesses to be developed that are consistent and comparable to the stiffness of the wall under service loads.
- permit loads to develop, that are consistent with the loads on the connectors in an actual wall

Serviceability limits in the new CSA code require that ties deflect less than 2 mm when loaded to 0.45 kN in tension or compression. For steel stud applications the CSA Masonry Connector Standard stipulates that these serviceability limit states incorporate tie pullout, flange rotation, and insulation deformation (if applicable) but exclude primary beam displacements. In the tests the contribution of the interior and exterior sheathing has not been included and it would be difficult to generalize this contribution. Moreover, the modeled support conditions (clamped conditions at 400mm) are probably on the conservative side. On the other hand any stiffening provided by sheathings, thermal insulation, etc. has been ignored . Clearly it is difficult to quantify the influence of a number of parameters, nonetheless these should not have a significant influence on the relative magnitude of the test results.

Finally it needs to be stated that practical and effective methods to remediate the tie-stud connection in existing BV/SS walls do exist. In this project only four possible alternatives were investigated. Clearly further developmental work is required, especially in better implementation of the Helifix Tie. This report does provide a basis for the assessment of other tie remediation methods or improvements to the four methods examined.