Strength and Stiffness Characteristics of Steel Stud Backup Walls Designed to Support Brick Veneer

Introduction

The use of brick veneer-steel stud (BV/SS) wall systems preceded adequate formal scientific investigation into its long-term serviceability and safety. Of particular concern to many parties was the performance of the walls under typical Canadian winter conditions. In response to this situation, Canada Mortgage and Housing Corporation (CMHC) sponsored a project in early 1986 to provide an independent investigation of BV/SS wall systems. The project was divided into three activities:

(i) the production of an Advisory Document on design and construction aspects;
(ii) the organization of a Canada-wide survey of BV/SS design and construction practices; and
(iii) laboratory testing of BV/SS systems and components.

The laboratory-testing component was further divided into five parts:

(i) fabrication and testing of components of steel stud backup walls;
(ii) fabrication and testing of brick masonry assemblies for leakage;
(iii) fabrication of a small wall test facility and tests of small walls for air, water vapour and heat flow;
(iv) tests of ties and interactions of ties with other wall components; and
(v) fabrication and tests of full-scale walls.

This report outlines the findings of Part 1 of the laboratory-testing program. The primary goal of the research was to document and evaluate the strength and stiffness characteristics of various components of the steel stud backup wall assembly. The test program had the following goals:

(i) documentation of the bending, torsional and web crippling strengths, as well as the deformational behaviour, of steel studs;
(ii) provision of data on strength, stiffness and construction features for top and bottom steel stud-to-track connection details;
(iii) evaluation of the effectiveness of various currently used types of bridging and bridging connections;
(iv) determination of the bracing capacity of gypsum board, as well as other sheathing materials; and
(v) observation of effects of cyclic loading and wetted gypsum board on the stiffness of the backup wall.

Research Program

The experimental test program was divided into two distinct phases: testing of steel stud-to-track connections and testing of full-scale backup wall panels.

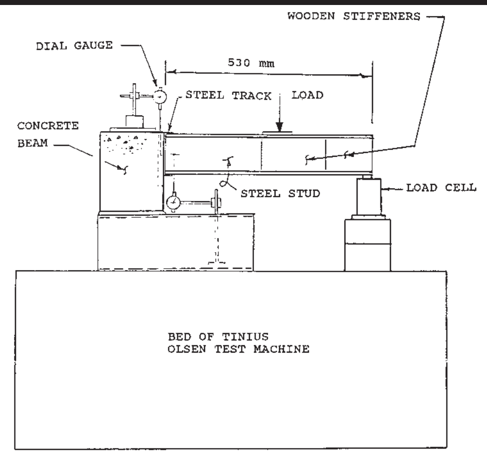
Steel Stud-to-Track Connections

A simple test set-up was devised to allow investigation and documentation of the strength and behaviour of various steel stud-to-track connection details. A short section of steel stud was fastened to a length of track using a specified fastening detail. The track was fastened to a concrete beam, which simulated a typical floor slab, so that the stud was horizontal. The free end of the stud was supported by a load cell in order to obtain the lateral force at the track (Figure 1). The whole apparatus fitted into an hydraulic test machine, which applied a lateral load to the top flange of the steel stud. Wooden stiffeners were inserted into the steel studs at the load location to prevent premature failure of the studs from application of the load.

Normally, each specimen was loaded in 500 Newton increments, but in some tests 250 Newton increments were used. The load head was lowered at a rate of 0.15 inches

Cette publication est aussi disponible en français sous le titre : *Résistance mécanique et rigidité des murs de fond à ossature d'acier conçus pour servir d'appui aux placages de brique*

Figure 1:
Experimental Set Up Used for Steel Stud to Track Connection Tests



per minute, increasing to 0.20 inches per minute as the loading approached the ultimate value. At the end of each load increment, deflection readings were recorded. Failure was defined as the point where the load dropped off significantly and displacement increased.

Test specimens were fabricated from cold-formed track and channel-shaped members. A total of 109 connections were tested. Parameters varied, including the size and thickness of the steel stud and track, the number of screws used to make the connection and the amount of gap left between the end of the steel stud and the inside face of the track. In addition, welded connections and a variety of movement connections were tested. In the last 11 tests, nail anchors, similar to those typically used in construction, were used to fasten the track to the concrete beam. In all other tests, the track was bolted to threaded roads anchored to the concrete beam.

Full-Scale Backup Wall Assemblies

The objective of this part of the research program was to document the influence of various factors on the strength and stiffness of the complete steel stud backup wall assembly. The strength and deformation characteristics of full-sized steel stud backup wall assemblies were evaluated using bending tests. The backup wall assemblies were installed horizontally into a test frame, and each steel stud in the backup wall panel was symmetrically loaded with two equal concentrated loads located approximately at the quarter points of the span. Beam tests were performed to evaluate the moment capacity of the studs. These results were then compared with the full-scale wall tests.

A total of 44 steel stud wall panels were fabricated and tested. The height, spacing and depth of the selected wall studs represented those normally used for residential construction. The varied parameters included the thickness of stud, number of rows of bridging and type of bridging connection details.

In addition, two analytical wall models were used to examine the influence of various features of steel stud backup walls on the overall behaviour of BV/SS wall systems. The findings of the experimental work were incorporated into the analytical models as input for some structural properties. The influence of the following variables were examined: stud stiffness, top track stiffness, bottom track stiffness, brick veneer stiffness, tie stiffness, top of brick restraint and wind-loading condition.

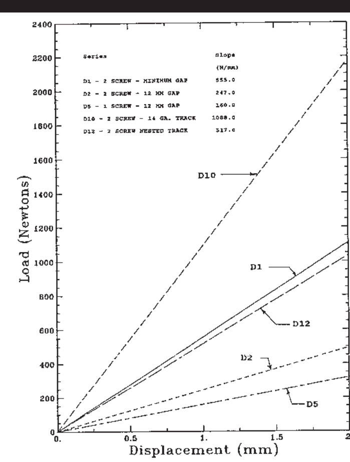
Results

Steel Stud-to-Track Connections

From an examination of the load-displacement relationships (Figure 2), it is evident that the type of fastening detail greatly affects the stiffness of the connection. A stiffer track was observed to stiffen the connection significantly compared to a stud and track of the same gauge. The connection details with two screws were much stiffer than the one-screw connections. However, the welded and the clip angle type of connection details were found to provide the stiffest type of connection detail.

In all of the tests, some stud end displacement was recorded, but the mode of failure was typically by web crippling of the stud. Web crippling occurs due to the fact that the steel stud must transfer the wind load to the supporting top and bottom tracks and, if the resulting end reactions are large enough, the local stress concentrations can cause web crippling. The experimental and predicted ultimate loads for web crippling at the stud end were compared for all the specimens. Predicted loads were calculated using an equation provided in CAN3-S136-M84, “Cold Formed Steel Structural Members.” The experimental values were greater than the predicted values for all cases.

Figure 2:
Load vs. Displacement Summary
20 Gauge, 90 mm Deep Steel Stud to Track Connection Tests



The specimens that used some type of clip angle to connect the web of the stud to the track did not fail by web crippling. Rather, the clip angle connection failed due to shearing of one of the two bolts connecting the clip angle to the stud or by the nail anchors pulling out of the concrete beam. The flexible clip angle failed by twisting of the flexible clip angle or by one of the nail anchors pulling out. The specimen with the welded connection failed due to web crippling caused by the loading head of the test machine.

Failure of the nail track fasteners occurred by pullout or bending or a combination of both. When failure occurred, it was usually noted that a cone of concrete surrounding the anchor was removed with the fastener. In three of the eleven tests, the nail penetration was inadequate, due to bending of the anchor by a stone upon entry into the concrete or due to inadequate depth of penetration. When the track anchors were spaced at 1,500 mm (vs. 900 mm) it was found that a greater track deflection occurred and, in some cases, the top flange of the track buckled before the stud failed by web crippling.

Full-Scale Backup Wall Assemblies

Failure of the steel stud backup wall panels was generally initiated when one or more studs started to twist significantly. Failure was always observed to occur in the region around one or more web cut-out holes. Visual examination of the panel immediately after testing indicated that no significant flange deformation or stud web crippling occurred at the stud-to-track connection.

The full moment resisting capacities of the studs were developed only when the studs were fully braced. Gypsum board sheathing attached to both flanges of the steel stud satisfied the full bracing requirement only if the gypsum was fastened at 150-mm centres or, if the gypsum was fastened at 305-mm centres and mid-span bridging was installed. In general, as the number of lines of bridging increased, there was a corresponding increase in the load-carrying capacity of the steel studs. Steel stud rotations also decreased significantly with the addition of the steel bridging. However, some of the composite action of the gypsum board was lost when the wall panels were subjected to cyclic loading.

Wall panels that were sheathed on the tension face of the steel studs showed an increase of only 6 to 10 per cent in moment over similar assemblies with no sheathing. However, even with mid-span bridging, walls sheathed on one side only were not sufficiently braced, exhibiting 60 per cent or less of the maximum moment capacity. Polystyrene insulation board on the compression face provided some bracing for the stud but not sufficient to develop the full expected moment capacity of the steel studs.

The bracing capacity of the gypsum was significantly reduced, and the full flexural capacity of the steel stud was not achieved when the gypsum was wetted by spraying a fine mist of water on the surface for a period of 12 hours before testing. After drying for 24 hours, some increase in panel stiffness occurred but quickly disappeared after a few more load cycles.

Preliminary tests with clips thinner than 16 gauge showed that significant clip bending occurred. When the screws were set closer to the bend in clip angle, less bending of the clip angle occurred. The welded connection provided the stiffest connection and should be considered for deeper and heavier steel studs. The locations of tie loads on the flange of the steel stud were found to affect the capacity of the steel stud significantly.

Analysis of BV/SS Wall Systems

The influence of various features of steel stud backup walls on the overall behaviour of BV/SS wall systems was examined analytically. When a BV/SS wall was subjected to out-of-plane wind forces, normal flexural tensile stresses developed at the bed joints. Once the tensile stresses exceeded the bond strength between the masonry unit and the mortar, cracking occurred.

A general observation in the analysis was that a large component of the out-of-plane displacement in the uncracked wall resulted from translation of the top track. Once the wall cracked, the deflection at mid-span was found to increase significantly. Before the brick veneer cracked, the top tie was heavily loaded. When the brick veneer cracked, a redistribution of tie loads occurred, and it was found that the ties near the mid-span of the wall became more heavily loaded. Cracking of the brick veneer occurred at approximately mid-height. After cracking, the steel stud flexural stresses increased by approximately 2.5 times over those in the uncracked wall. This indicated that the brick veneer no longer carried the greater portion of the wind load. The allowable stud deflection to prevent the crack width from exceeding 0.1 to 0.2 mm and to prevent significant water ingress into the wall was determined to be L/1,800 and L/900 respectively.

Increasing the bottom track stiffness resulted in only a minor increase in the wind load required to cause brick veneer cracking. The stiffness of the top steel stud-to-track connection greatly influenced the overall out-of-plane deflections of the BV/SS wall system but had very little influence on the reduction in brick veneer stress. Increasing the stiffness of the steel stud by 50 per cent reduced the stress on the brick veneer by 15 per cent to 23 per cent and decreased the out-of-plane deflections. Reducing the stiffness of the brick veneer caused the steel stud backup wall to share a greater portion of the total lateral wind load, which in turn reduced the brick veneer stress by 8 per cent to 30 per cent. Increasing the brick tie stiffness resulted in an increase in the brick veneer stress. As the stiffness of the restraint at the top of the veneer increased, the brick veneer stress increased and the out-of-plane deflections decreased significantly.

Implications for the Housing Industry

This research indicated that any design criteria for BV/SS wall systems based solely on limiting maximum deflection of the steel stud is not satisfactory, because deformations and displacements of the ends of the steel studs in the track are also significant. The design of the BV/SS wall system should take the following into consideration:

(i) brick veneer and backup wall interaction;
(ii) stiffness of the top and bottom track connection detail;
(iii) steel stud stiffness;
(iv) tie stiffness;
(v) brick veneer stiffness; and
(vi) flexural bond strength.

The designer must also consider the stiffness of the backup wall to prevent the brick veneer from cracking and must minimize the width of the crack to control water penetration.

For all test specimens, the ratio of the experimental failure load to the ultimate predicted load, using a resistance factor of 1.0, was found to be greater than one. Thus, for designing steel stud backup walls, the equations provided in CAN3-S136 can be used for evaluating web crippling potential.

The study also revealed that, unless the integrity of the sheathing can be guaranteed over the life of the structure, some other form of bracing must be provided. This is usually accomplished by bracing the steel studs with steel bridging. If steel bridging is used, a maximum spacing of 1,200 mm between braces is recommended to control the amount of stud twisting and to prevent premature failure. The ends of any type of steel bridging should be adequately anchored. This is particularly important for notched face bridging, which was found to be ineffective unless the ends of the bridging were adequately fastened. It was also concluded that, to minimize bending of clip angles, clips should be 16-gauge or thicker and screws should be placed no further than one third of the length of the leg away from the bend in the angle.

Premature failure at web cut-out holes can be prevented by not allowing additional web cut-out holes in regions of high combined stresses, except at the brace point locations on the steel stud. If additional web cut-out holes are required for services at locations other than at the bracing points, it is suggested that they be located 300 to 400 mm from either end of the steel stud. In addition, no brick ties, which induce web crippling, should be located directly over these holes.

Lastly, the report recommends that anchor spacings of 800 mm on centre or less are good practice regardless of the type of anchors used.

The results of this work have been integrated into the following publications:

Exterior Wall Construction in High-Rise Buildings (for engineers)
Best Practice Guide: Brick Veneer Steel Stud (for architects)

Project Manager: *Jacques Rousseau*

Research Consultant: *Department of Civil Engineering and Engineering Mechanics, McMaster University*

Research Report: *Strength and Stiffness Characteristics of Steel Stud Backup Walls Designed to Support Brick Veneer, 1991*

A full report on this research project is available from the Canadian Housing Information Centre at the address below.

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