Study of Perforated Steel Wall Studs

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ABSTRACT

In this paper, the behaviour of a gypsum-sheathed perforated steel wall stud is studied. The stub column tests were conducted to evaluate the local buckling and distortional buckling behaviour of the stud sections. In the wall stud assembly tests, the overall system behaviour was studied with studs, tracks and gypsum boards assembled as in the actual construction. The bending moment resistance of the section was studied in bending tests for the sections without wallboards. The experimental results were compared with the design code predictions using Eurocode 3, Part 1.3 and the draft of the Australian Standard for cold-formed steel structures.

INTRODUCTION

The investigated structure is a part of the EKO-PRO- wall structure system developed by the company Aulis Lundell Oy. The structure is normally used as a structural component in small houses. The structure consists of cold-formed sigma-sections as studs and U-sections as tracks and gypsum wallboards attached to the stud flanges. The studs and tracks are made from structural steel S350GD+Z. The section height of the studs and tracks is 145mm or 195mm and thickness varies between 0.7mm to 1.5mm. The web parts of the stud and track are slotted to eliminate the cold bridge effect. The slotted thermal stud offers a considerable improvement in thermal performance over the solid steel stud. The resistance through the perforated stud is only slightly lower than through the wood stud. A schematic picture of the wall structure is shown in Figure 1. Furthermore, the system incorporates end stiffeners installed into the end of the studs for improving shear and local transverse resistance of the web.

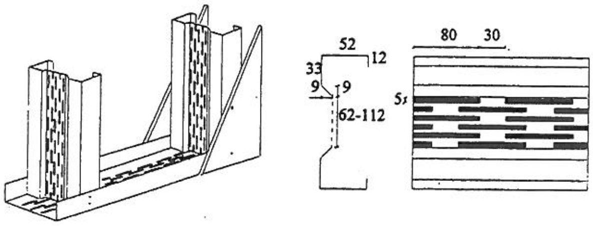

Figure 1: The wall structure and section

EXPERIMENTAL RESEARCH

Stub Column Tests

Test specimen and procedure

The lengths of the stub column specimens varied between 1 050- and 1 150mm, and the gypsum wallboards (width 600mm) were attached to the stud flanges at 300mm spacing. The ends of each stub column were machine-ground and hand-filed to ensure a flat and plane surface perpendicular to the specimen's longitudinal axis. The stub columns were centred in a 500kN hydraulic testing machine. The stub columns were simply placed between the loading plates without using welding or grouting. The aluminium plates of thickness 1.5mm were placed between specimens and the loading plate to ensure uniform stress distribution. The position of the upper loading plate was corrected in case of an observed out-of-parallelism of the specimen ends. The length of the specimens was chosen long enough to investigate distortional buckling as well.

Test results

The specimens failed in distortional buckling. In the failure stage the wallboard fasteners pulled through the sheathing. The experimental ultimate loads for the stub column specimens are given in Table 1. The experimentally determined ratios of the effective area to the gross area of the section are also indicated as measured yield stress.

TABLE 1

STUB COLUMN TEST RESULTS

Specimen	Length [mm]	Yield stress [N/mm^2]	Ultimate load [kN]	Ag [mm^2]	Aeff [mm^2]	Aeff/ Ag
SCB-145-1.0-1	1048	402	51.1	215.5	127.1	0.590
SCB-145-1.0-2	1048	402	52.3	215.5	130.1	0.604
SCB-145-1.3-1	1046	369	66.9	275.3	181.3	0.659
SCB-145-1.3-2	1046	369	71.5	276.4	193.8	0.701
SCB-195-1.0-1	1159	402	49.1	236.2	122.1	0.517
SCB-195-1.0-2	1160	402	49.4	236.2	122.9	0.520
SCB-195-1.3-1	1158	369	71.4	303.4	193.5	0.638
SCB-195-1.3-2	1158	369	65.0	303.3	176.2	0.581

Note: Specimen designation: SCB-195-1.3-1. SCB= stub column with board, 195= section height (mm), 1.3= thickness (mm), 1= test number. A_g = Gross cross section, A_{eff} = F/f_y =Effective cross section.

Bending Tests

Test specimen and test set-up

The test specimens were constructed of two equally sized sigma-sections of length 3 500mm or 4 000mm. The sections were placed facing each other in a box-beam arrangement, as shown in Figure 2. This configuration was used to avoid the shear centre eccentricity problem. The sections were interconnected by steel angles, 20x20x2mm, attached to the flanges using 3.5mm self-drilling screws. Angles were located in both flanges at 300mm spacing. Angles were not used in the constant moment region to allow for the free distortional buckling mode for the upper flange.
The test specimens were simply supported and subjected to two point loads, as shown in Figure 2. All loads and reactions were transferred by 80x5mm loading plates to avoid localized crippling of the webs. Lateral supports were used near the loading points on both sides of the beam.

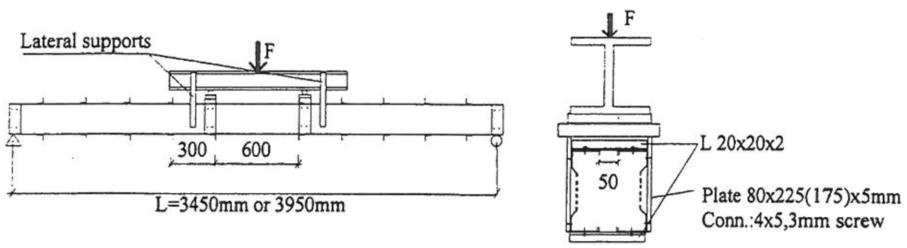

Figure 2: Test set-up for bending test

Test results

The test specimens failed by distortional buckling of the upper flange at the constant moment region. Test specimen BE-195-1.5 failed by lateral buckling between the end support and the lateral support. Distortional buckling mode was observed at this stage. The experimental bending moment resistances of the sections are shown in Table 2. The test results of the tensile tests are also shown in Table 2.

TABLE 2

TEST RESULTS OF BENDING TESTS

Specimen	Length [mm]	Yield stress [N/mm^2]	Failure moment [kNm]
BE-145-1.0	3450	412	2.65
BE-145-1.5	3950	405	5.42
BE-195-1.0	3450	412	3.92
BE-195-1.5	3950	405	6.80

Wall Stud Assembly Tests

Test specimen

The wall stud assemblies consisted of two 2 600mm-long studs spaced at 600mm. Two 1 200mm-wide gypsum wallboards of thickness 9mm and 13mm were attached to the stud flanges at 300mm spacing but without being attached to the tracks. Tracks and end stiffeners were also used in the assemblies. The specimen is shown in Figure 3.

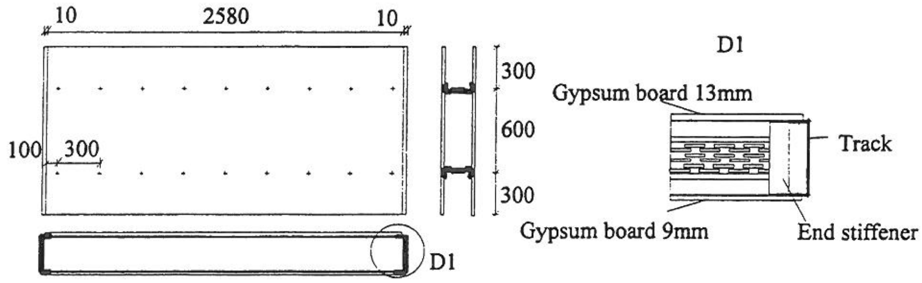

Figure 3: Wall stud assemblies

Test procedure

The test set-up is shown in Figure 4. The wall assemblies were subjected to combined compression and lateral loads. The axial compression load was applied with 200kN jacks. The wall assemblies were pinned (about the strong axis) at both ends and an eccentricity of h/5 was used for the top end. The loading rates used were 1.0kN/min (t=0.7mm), 1.5kN/min (t=1.0mm) and 2.0kN/min (t=1.3mm). Lateral supports were also used to give better control of the assembly at a failure moment. Lateral loading was arranged with weights on the wall assembly. Deflections of the assembly were measured with six LVDT transducers. The movement of the jack was also measured to record the total compression of the wall assembly. Acceptance, strength and failure tests were conducted for all assemblies according to Eurocode 3, Part 1.3 (1996). The vacuum loading was chosen as the most critical loading case and a 13mm gypsum board (used inside the wall structure) was used in tests on the top of the assembly.

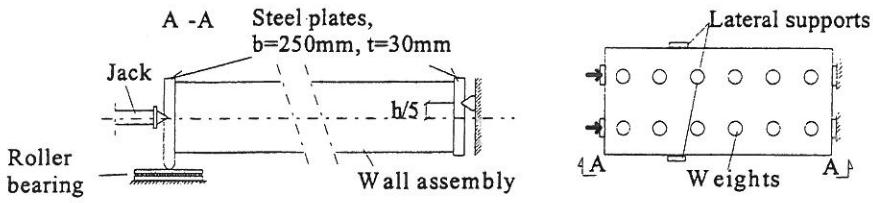

Figure 4: Test set-up for wall stud assembly test

Test results

All test specimens failed by distortional buckling of the upper flange. The wallboard fasteners pulled through the sheathing at the failure moment. At this stage the distortional mode was already considerable. All failures occurred within maximum moment region except the concentric loaded specimen WA-195-1.0-2 and the specimen WA-145-0.7-2 failing at a region where one fastener was lacking. Test results are shown in Table 3. Test results of the tensile tests are also shown in Table 3. The last column indicates the failure position from the eccentric loaded end.

TABLE 3

TEST RESULTS (PER ONE SECTION) OF THE WALL ASSEMBLY TESTS

Specimen	Yield stress [N/mm^2]	Lateral load [kN/m]	Eccentri-city [mm]	Max. moment [kNm]	Failure load [kN]	Failure pos. ecc. end [mm]
WA-145-0.7-1	390	0.55	29	0.732	13.1	630
WA-145-0.7-2	390	0.30	29	0.542	15.1	1820
WA-145-1.0-1	402	0.66	29	1.000	23.0	330
WA-145-1.0-2	402	0.66	29	1.033	24.5	360
WA-145-1.3-1	369	0.66	29	1.254	35.3[1]	360
WA-145-1.3-2	369	0.38	29	1.305	45.0	570
WA-195-0.7-1	390	0.55	39	0.866	15.7	850
WA-195-0.7-2	390	0.30	39	0.691	16.5	460
WA-195-1.0-1	402	0.66	39	1.400	31.4	380
WA-195-1.0-2	402	0.66	0	0.627	38.0	2150
WA-195-1.3-1	369	0.66	39	1.721	42.1	380

[1] Loading removed at stage 35.0kN. In reloading the specimen failed at 35.3kN.

ANALYTICAL METHODS

Elastic Distortional Buckling

Because of perforation of the web, the transverse bending stiffness of the section is very low and the section is very sensitive to the distortional buckling under compression and bending. In distortional mode of buckling, the edge-stiffened flange elements of the section deform by rotation of the flange about the flange-web junction. The distortional buckling mode occurs at longer wavelengths than local buckling. The distortional buckling modes are shown for the perforated sigma-section in compression in Figure 5.

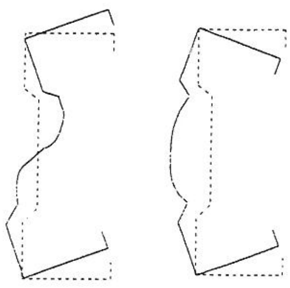

Figure 5: Distortional buckling modes for sigma-sections

The distortional buckling stresses for the sections were determined using the finite strip method (FSM) and finite element method (FEM). In the models, the perforated web was modelled as a plain plate with reduced shear modulus and modulus of elasticity. The shear modulus value of $G_c = 1\ 000 N/mm^2$ and elastic modulus value of $E_c = 8\ 000 N/mm^2$ were used. Graphs for the buckling stresses versus buckle half-wave lengths for a perforated sigma-section 145-1.3 in compression and bending are shown in Figure 6. The buckling stresses were determined in cases where the flange is laterally unrestrained or restrained. The buckling stresses were determined using THIN-WALL (1996) computer program based on the finite strip method.

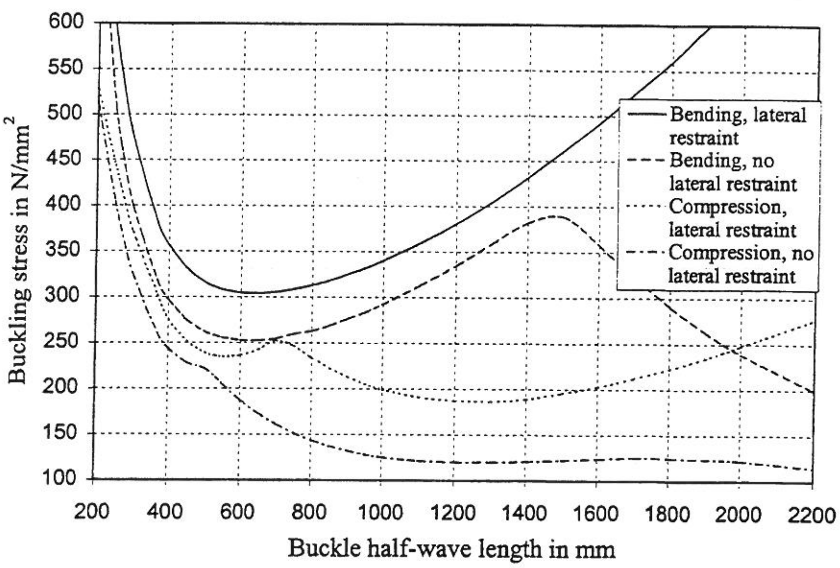

Figure 6: 145-1.3-section: buckling stress versus half-wavelength

The results of the FSM analysis are based on simply supported end boundary conditions. This method is applicable to longer sections where multiple buckle half-waves occur in the section length (Hancock et al. 1994). Because the stub column tests were conducted between rigid end-platens, FEM analyses were used to take into consideration the end boundary conditions. The results of both buckling analyses for section 145-1.3 are presented in Figure 7. The abscissa in Figure 7 for the FSM analysis indicates one half-wave and for the FEM analysis the abscissa indicates real column length between the rigid end-platens. In the FEM analysis curve, the number of buckling half-waves and buckling mode (a = asymmetric and s = symmetric distortional buckling mode) are marked in Figure 7. In the FSM analysis curve, the first minimum is asymmetric and the second minimum symmetric distortional buckling mode. It can be observed that the FEM results approach the FSM results when the column is long enough and the section buckles in multiple distortional half-waves.

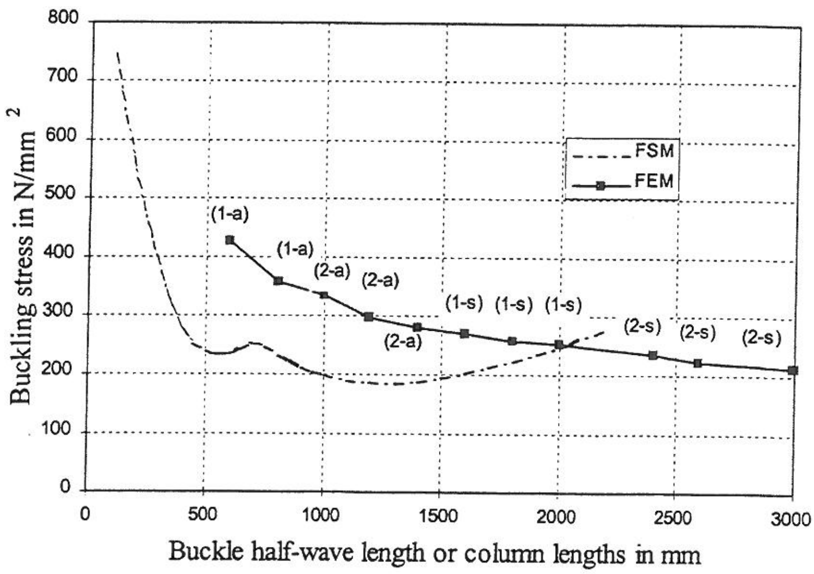

Figure 7: Results of FSM and FEM analyses

Elastic Flexural Buckling

The web perforation decreases the flexural buckling about the steel stud strong axis due to shear deformations. For a simply supported column with bending stiffness EI, shear stiffness S_v and length L the buckling load $N_{cr,id}$ obtained by the Timoshenko shear beam theory (Gjelsvik 1991) is

$$N_{cr,id} = \frac{N_e}{1+\frac{N_e}{S_v}} \qquad (1)$$

where N_e is the Euler load for the column ignoring the effect of shear deformation. Allen (1969) has presented buckling load formula for sandwich structures with thick faces when the bending stiffness of the thick faces is considerable compared to that of whole structure. This sandwich theory was applied also for perforated steel studs. The buckling load can be expressed as

$$N_{cr.id} = N_e \left(\frac{1+\frac{N_{ef}}{S_v}-\frac{N_{ef}}{S_v}\frac{N_{ef}}{N_e}}{1+\frac{N_e}{S_v}-\frac{N_{ef}}{S_v}} \right) \tag{2}$$

where N_{ef} is the Euler buckling load of one flange. In the case of a perforated steel stud N_{ef} is determined for the part of the section that is outside the perforated area (plain section flange).

Design Methods

Modified Eurocode 3 method

In Eurocode 3 (1996), the distortional mode of buckling is taken into account by assuming edge or intermediate stiffeners as a compressed strut on an elastic foundation where elastic foundation is represented by a spring depending upon the bending stiffness of adjacent parts of plane elements and on the boundary conditions of the element. The design strength is based upon design curve *a* with α = 0,13. In design the reduced thickness of the stiffeners is used. This method gives very conservative compression strength values if reduced bending stiffness of the web is used. The method does not also consider the length of the section. In this research, the Eurocode 3 method was modified by determining critical stresses for stiffener by FSM or FEM stability analyses. Local buckling was considered using effective widths according to Eurocode 3. The perforated web was ignored and the stiffener between the perforated part and plain part was assumed as an edge stiffener. The interaction between local and global buckling was checked according to Eurocode 3. The shear deformations were considered in flexural buckling. The compression and bending moment interaction formula was simplified.

Australian Standard method

In the draft of Australian Standard (1996) the distortional buckling strength is checked by using separate design curves. The compression capacity of the section is lesser of the distortional buckling strength or interaction of local and overall buckling. A design method for computing the distortional buckling bending moment resistance of flexural members is also presented in the standard. The design methods were used as such using distortional buckling stresses determined by FSM and FEM. The same compression and bending moment interaction formula was used as in the modified Eurocode 3 method.

The effect of wallboards

In both standards, the structural contribution of the attached sheathings is ignored and the calculated member resistance is solely based on the condition of lateral bracing of the member, or on the bare steel frame approach. According to the Australian Standard, the sheathing can be assumed to give a lateral and rotational support to the stud in the plane of the wall, provided that the stud, sheathing and attachments satisfy certain requirements. In this research, the sheathing is assumed to give lateral support in the plane of the wall. The gypsum board connection gives also a rotational support to the stud, which considerably improve the distortional buckling strength. However, the connection can not fully restrain the rotation of the flange and the lip and thus it can not form a nodal point for buckling. The influence of a rotational restraint was examined ignoring the restraint and assuming the nodal point to connection. The bending stiffness of the sheathing was ignored.

COMPARISON OF CALCULATED AND TEST RESULTS

Stub Column Tests

A comparison of the test and calculated values is shown in Table 4. The elastic distortional stresses were determined using FEM to take into account the end boundary conditions. The sheathing was assumed to give lateral support in the plane of the wall and the rotational support of the gypsum board connection was ignored. The calculated values are unconservative about 3% when using the modified Eurocode 3 method and about 9% when using the Australian Standard. The main reason for these unconservative results is the difference between the end boundary conditions in the tests and in the FEM model. The warping of the section was not fixed in the tests, but the section was simply placed between rigid platens without welding or grouting.

TABLE 4

COMPARISON OF TEST AND CALCULATED VALUES OF STUB COLUMNS

Test specimen	Failure load N_t [kN]	EC3 N_p [kN]	EC3 N_t/N_P	AUS N_p [kN]	AUS N_t/N_P
SB-195-1.3-1	71.4	77.5	0.92	76.7	0.93
SB-195-1.3-2	65.0	77.5	0.84	76.7	0.85
SB-145-1.3-1	66.9	73.4	0.91	72.4	0.92
SB-145-1.3-2	71.5	73.4	0.97	72.4	0.99
SB-195-1.0-1	49.1	50.6	0.97	54.6	0.90
SB-195-1.0-2	49.4	50.6	0.98	54.6	0.90
SB-145-1.0-1	51.1	47.8	1.07	54.0	0.95
SB-145-1.0-2	52.3	47.8	1.09	54.0	0.97
Mean value			0.97		0.92

Bending Tests

In the bending moment capacity calculations, elastic distortional buckling stress was used corresponding to the half-wave-length of 600mm, which was the free moment span in the tests. In the case of test specimen BE-145-1.5 the minimum stress occured in the curve at a half-wave length of 500mm. The capacities were calculated according to the modified Eurocode 3 and the draft of the Australian standard, similar to the stub column calculations. The same stress was used for both stiffeners in the upper flange. Comparisons with the experimental and calculated capacities are shown in Table 5. These results based on a small number of tests seem to indicate that both of these two methods predict the moment capacity rather well.

TABLE 5

COMPARISON OF TEST AND CALCULATED VALUES OF BENDING TEST SPECIMENS

Test specimen	Failure moment M_t [kNm]	Elastic distortional stress [N/mm^2]	EC3: M_P [kNm]	EC3 M_t/M_P	AUS: M_P [kNm]	AUS M_t/M_P
BE-195-1.5	6.80	268	7.43	0.92	6.99	0.97
BE-145-1.5	5.42	355	5.65	0.96	5.31	1.06
BE-195-1.0	3.92	177	3.63	1.08	3.60	1.09
BE-145-1.0	2.65	229	2.73	0.97	2.62	1.03
Mean value				0.98		1.04
St. dev.				0.07		0.05

Wall Stud Assembly Tests

The test-to-predicted interaction ratios of moment and compression capacities are presented in Table 6. The following simplified clause for interaction has been used:

$$\frac{N_{Test}}{N_P} + \frac{M_{Test}}{M_P\left(1 - N_{Test} / N_{cr.id}\right)} \tag{3}$$

where $N_{cr.id}$ is the Euler buckling load including shear deformations according to Eqn. (2). In the EC3 column of Table 6, the compression and bending capacities were determined by modifying Eurocode 3. The elastic distortional stresses were determined using FSM and by assuming a lateral support and ignoring the rotational support. In the EC3 (stub) column in Table 6, the effective cross-section area is taken from the stub column tests for the compression capacity calculation. In the next column, the resistances were calculated assuming that gypsum board fasteners guarantee the full rotational restraint and the half-wave of the buckle is the fastener spacing (300mm). In the AUS column, the compression and bending moment capacities were determined according to the draft of the Australian Standard ignoring the rotational support and in the last column assuming a full rotational support, as in the Eurocode 3 method.

Both calculated strength values of the studs are about 20% conservative, if a lateral support is assumed in the wall plane and the perpendicular direction support is ignored so that distortional buckling can be freely developed. The modified Eurocode 3 method gives slightly unconservative values if the full rotational support is assumed and a buckling length of 300mm is used. The Australian Standard gives more unconservative values though in this case. The test values are quite consistent with the test results when the effective cross-section area was determined from the stub column tests.

TABLE 6

COMPARISON OF TEST AND CALCULATED VALUES OF WALL STUD ASSEMBLIES

Test specimen	Failure load N_t[kN]	Failure moment M_t[kNm]	EC3	EC3 (stub)	EC3 (L=300)	AUS	AUS (L=300)
195-1.3-2r-1	42.1	1.721	1.15	1.06	0.95	1.18	0.87
195-1.0-2r-1	31.4	1.400	1.35	1.16	1.04	1.27	0.98*
195-1.0-2r-2	38.0	0.627	1.31	1.08	0.99	1.22	0.91*
145-1.3-2r-1	35.3	1.254	1.07	1.00	0.94	1.09	0.87*
145-1.3-2r-2	45.0	1.305	1.31	1.22	1.15	1.32	1.05*
145-1.0-2r-1	23.0	1.000	1.12	0.97	0.92	1.06	0.84*
145-1.0-2r-2	24.5	1.033	1.18	1.03	0.97	1.12	0.89*
Mean value			1.21	1.07	0.99	1.18	0.92
St. deviation			0.11	0.09	0.08	0.10	0.07

*Compression capacity determined by interaction of local and global buckling ($N_c = A_e f_n$)

CONCLUSIONS

In this paper, the behaviour of a gypsum-sheathed perforated steel wall stud was studied. The stub column tests were conducted to evaluate the local buckling and distortional buckling behaviour of the stud sections. The bending moment resistance of the section was studied in bending tests for the sections without wallboards. In the wall stud assembly tests, the overall system behaviour was studied with studs, tracks and gypsum boards assembled as in the actual construction. The web perforated stud section is very sensitive to the distortional buckling mode. The gypsum board connection improves the buckling resistance but the connection can not fully restrain the rotation of the flange and the lip. The calculated strength values are about 20% conservative for the interaction of compression and bending moment if the stud is assumed laterally braced and rotational support of the fasteners is ignored. Design calculations for perforated steel studs can be made, instead of design by testing, with sufficient reliability using these support conditions.

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