Bearing Strength of Slabs on Grade Supporting a Cold-Formed Steel Wall in Low-Rise Building

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Abstract: The main objective of this research project was to develop a better understanding of the bearing strength of slabs on grade supporting load-bearing walls made of cold-formed steel studs and tracks used in residential and commercial multistory constructions. A total of 60 specimens were manufactured with four different configurations of stud-track assembly: single stud, single-stud wall, back-to-back, and back-to-back wall. The test results showed that the bearing strength was affected by the configurations of the stud-track assembly, and that the bearing strength gradually increased as the stud-track assembly was located at the inner side from the edge of the slab due to the confinement effect of the surrounding concrete. An analytical study using a finite-element model was performed to develop the analytical equations used to predict the bearing area of the stud-track assembly. Design guidelines were proposed based on the test results and the analytical study, which included the equations to compute the design bearing capacity of slabs on grade at varying distances from the edge of the slab to the stud-track assembly.

DOI: 10.1061/(ASCE)1076-0431(2006)12:1(24)

CE Database subject headings: Bearing capacity; Buildings, residential; Buildings, low-rise; Walls; Cold-formed steel; Concrete slab.

Introduction

The use of cold-formed steel in construction dates back to 1850s. A growing application of cold-formed steel framing is in midrise construction that is three to five story structures with load bearing wall assemblies. Fig. 1 shows a typical residential and commercial multistory structure during construction using cold-formed steel framing system. These wall assemblies are composed of structural members such as wall stud, track, bracing, sheathing materials, and fasteners. Currently, the *North American Specification for the Design of Cold-Formed Steel Structural Members* (AISI 2001) is used to determine the design of cold-formed steel members and connections.

Although the North American specification addresses the design of the cold-formed steel members and connections, it is silent regarding the design of the foundation upon which the steel structure is supported. Therefore, engineers are making assumptions to estimate the bearing capacity of slabs on grade-supporting load-bearing walls constructed of cold-formed steel. A typical load-bearing wall constructed of cold-formed steel is illustrated in Fig. 2. As shown in this figure, the load applied to the wall is transferred to the concrete slab through its studs and bottom track. As a result, the load is possibly applied to a limited area of the slab on grade as a form of concentrated load. Thus, it is important to evaluate the bearing capacity of the slab on grade. However, many have overlooked this key issue of concrete bearing capacity, as there is limited research on the topic to date. With steel studs becoming popular, especially in the past couple of decades, this probably needs immediate attention.

Under the current American Concrete Institute building code (ACI 318-02), the maximum bearing load of concrete is specified based on the test results of concrete block loaded through rigid plates (Hawkins 1968). However, it is doubtful whether it is appropriate to apply ACI 318-02 directly to a concrete slab supporting a cold-formed steel wall, particularly because of the nature of the equations and how they handle the confinement effect. The first study on this topic was reported by Trestain (1991) and resummarized by Peyton (2000). The purpose of Peyton's article was mainly to express the need for research into the problem. Trestain (1991) proposed an approximate method for calculating the bearing capacity of slabs on grade, which was also adopted in the current *AISI Cold-Formed Steel Framing Design Guide* (AISI 2002). However, both Trestain and Peyton suggested that the formula should only be used as an aid since it was not proven by any experimental studies. The first experimental study on the subject was conducted by Chin (2001) at the University of Manitoba, in which tests were conducted in order to determine if the full local buckling capacity of studs would be reached before the concrete of supporting slabs failed. The conclusions were that the full buckling capacity of the cold-formed steel studs could be reached when the members were located at a sufficient distance from the edge. This research, however, raised a question of how edge distance affects concrete bearing capacity.

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Note. Discussion open until August 1, 2006. Separate discussions must be submitted for individual papers. To extend the closing date by one month, a written request must be filed with the ASCE Managing Editor. The manuscript for this paper was submitted for review and possible publication on October 20, 2004; approved on April 27, 2005. This paper is part of the ***Journal of Architectural Engineering***, Vol. 12, No. 1, March 1, 2006. ©ASCE, ISSN 1076-0431/2006/1-24–32/$25.00.

Fig. 1. Typical residential and commercial multistory cold-formed steel framing system

Scope and Objectives of This Study

As a result of the studies mentioned and the need expressed by the designers, this research project was initiated to investigate the bearing strength of slabs on grade supporting load-bearing cold-formed steel wall through experimental and analytical studies. The objectives of this study were threefold: the first objective was to investigate the bearing strength of slabs on grade through the experimental program; the second objective was to evaluate the applicability of the ACI 318-02 approach and the recommendations proposed by Trestain through comparison with the test results; and the third objective was to propose design guidelines based on the test results and the supporting analytical study, which finally include design equations to determine the bearing strength of slabs on grade.

However, as a first step into this topic, this study only focused on plain concrete slabs with a minimum thickness (89 mm) that is likely seen in residential constructions. There are thicker reinforced concrete slabs with reinforced footing systems widely used in residential and commercial multistory building constructions. These constructions require further consideration of additional loading such as seismic, wind, or unexpected live loads according to the current codes and practices. In addition, effects of different soil conditions are of great concern for the design of slabs on grade. However, these are beyond the scope of this study.

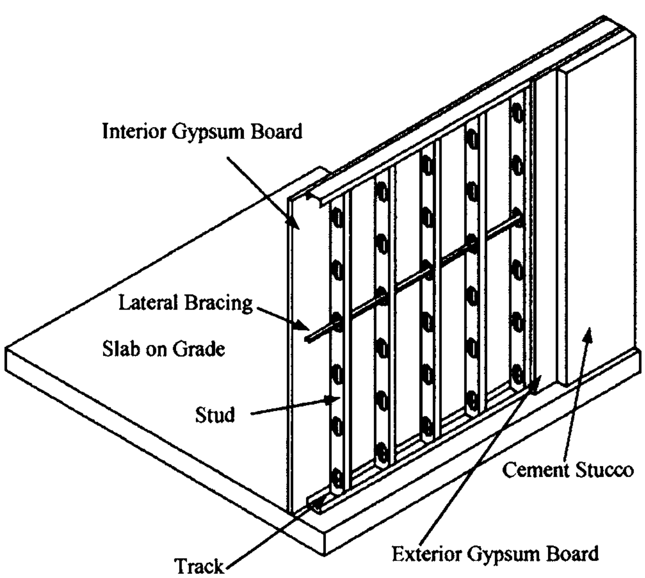

Fig. 2. Typical cold-formed steel wall

Experimental Program

A total of 60 specimens were tested for the parametric study as shown in Table 1. The specimens were constructed with four different configurations—single stud, single-stud wall, back-to-back, and back-to-back wall—as shown in Fig. 3. As shown in Fig. 3, for single-stud and back-to-back configurations, a stud-track assembly was placed on an 89-mm-thick concrete slab. Wire mesh of 4.8 mm in diameter was placed in each of slabs for shrinkage control. As a pilot study in this topic, the slabs thickness of 89 mm was selected to simulate the worst case that is likely seen in the residential building constructions. For single-stud wall and back-to-back wall configurations, two stud-track assemblies were placed on the concrete slab. Three different concrete strengths

Table 1. Test Matrix

Specimen	Configuration[a]	f'_c (MPa)	e^b (mm)	N^c
S-33-0	S	33	0	3
S-33-44	S	33	44	1
S-33-89	S	33	89	2
S-40-0	S	40	0	3
S-40-51	S	40	51	4
S-40-102	S	40	102	2
S-54-0	S	54	0	3
S-54-44	S	54	44	2
B-33-0	B	33	0	3
B-33-44	B	33	44	3
B-33-89	B	33	89	2
B-40-0	B	40	0	2
B-40-51	B	40	51	2
B-40-102	B	40	102	2
B-54-0	B	54	0	3
B-54-44	B	54	44	3
B-54-89	B	54	89	2
SW-40-0	SW	40	0	3
SW-40-51	SW	40	51	3
SW-40-102	SW	40	102	3
BW-40-0	BW	40	0	3
BW-40-51	BW	40	51	3
BW-40-102	BW	40	102	3

[a]Configuration of stud-track assembly (S=single stud; B=back-to-back; SW=single-stud wall; and BW=back-to-back wall).
[b]e=distance from the edge of the slab.
[c]N=number of specimens.

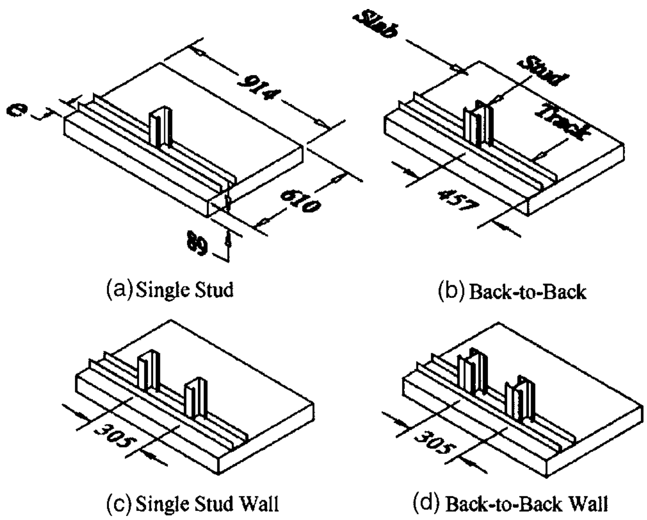

Fig. 3. Test specimen configurations (unit: mm)

were used for parametric study as shown in Table 1. Nominal cross-sectional dimensions for the stud and track are presented in Fig. 4. The studs in the stud-track assembly were cut to a height of 51 mm in order to minimize the occurrence of overall buckling and focus on slab performance. Distances from the edge of slabs to the outer side of the stud-track assembly (defined as *e* in Fig. 3), were varied to investigate the bearing strength of slabs on grade in relation to the distance.

The test setup shown in Fig. 5 includes an aggregate box (1,524 × 1,829 × 609 mm) filled approximately 304 mm deep with aggregate in order to simulate supporting soil. It was assumed that there was no flexural behavior of slabs induced by differential settlement of supporting soil, since the change of failure mode could result in the change in bearing-capacity measurement. A reaction beam supported by vertical steel rods and a hydraulic jack were utilized to apply the load. The applied load was measured by a 220-kN load cell. Thus, a great effort was made to keep the deflections constant in order to eliminate the chance of a flexural failure. A reaction beam was used to apply restraining loads to the slab on the three edges in order to prevent premature failure of the slab due to the flexure. The restraining loads effectively simulated the effect of an infinitely large slab on grade. The restraining load was applied by means of two screw

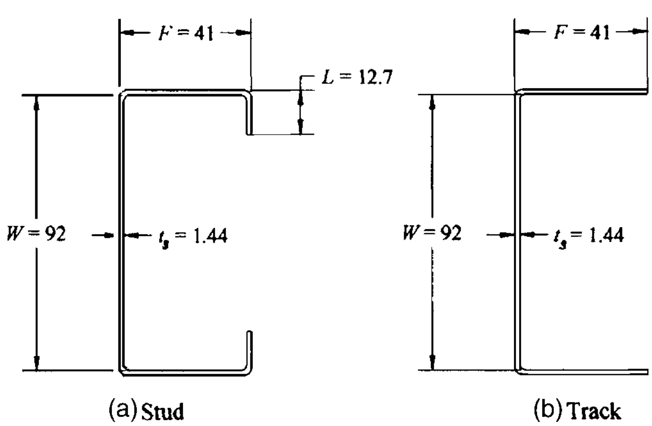

Fig. 4. Nominal cross-sectional dimensions of stud and track (unit: mm)

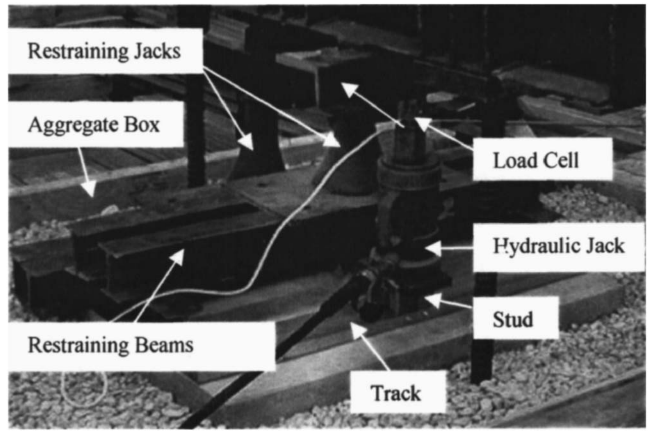

Fig. 5. Test setup and instrumentation

jacks and transferred to the slab through a series of steel beams as shown in Fig. 5. During testing, deflection was measured by two dial gauges, located at the center and corner edge of the slab as shown in Fig. 6.

Once the slabs were positioned in the aggregate box, the restraining jack was tightened to apply the pressure so that the slab was secured firmly. The hydraulic jack was then used to apply the load to the slab through the stud-track assembly. The slab was loaded at approximately 1.3 kN/s. While the load was being applied to the slab, the dial gauges were monitored constantly to ensure that differential deflection did not occur. As the deflection in the middle of the slab increased, the restraining jack was also tightened so as to keep the deflection constant throughout the width of the slab. The restraining jack controlling the restraining beams opposite the load edge was also advanced throughout the tests to ensure constant deflection throughout the length of the slab. Each slab was loaded up to failure. The loads at initiation of crack and failure were recorded for each of the tests. Failure was defined by one of two situations: either when the slab would not take any more loads or when the crack reached a width of approximately 1.3 mm. Typical failure modes of the tested slabs are presented in Fig. 7. As shown in Fig. 7, the test slabs failed due to a combination of flexure and shear.

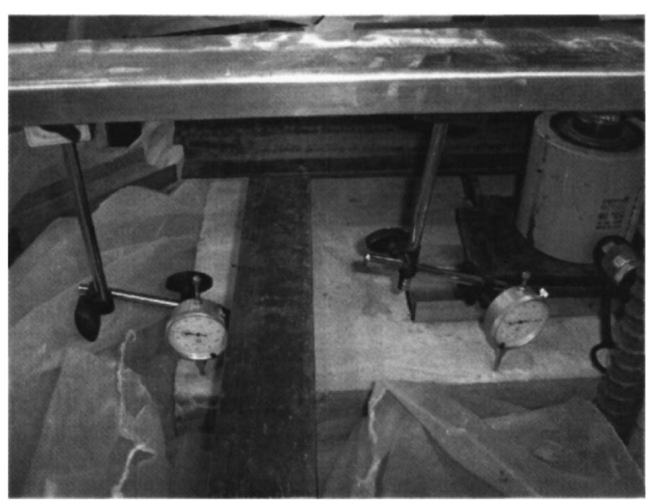

Fig. 6. Location of dial gauges used to control differential deflection

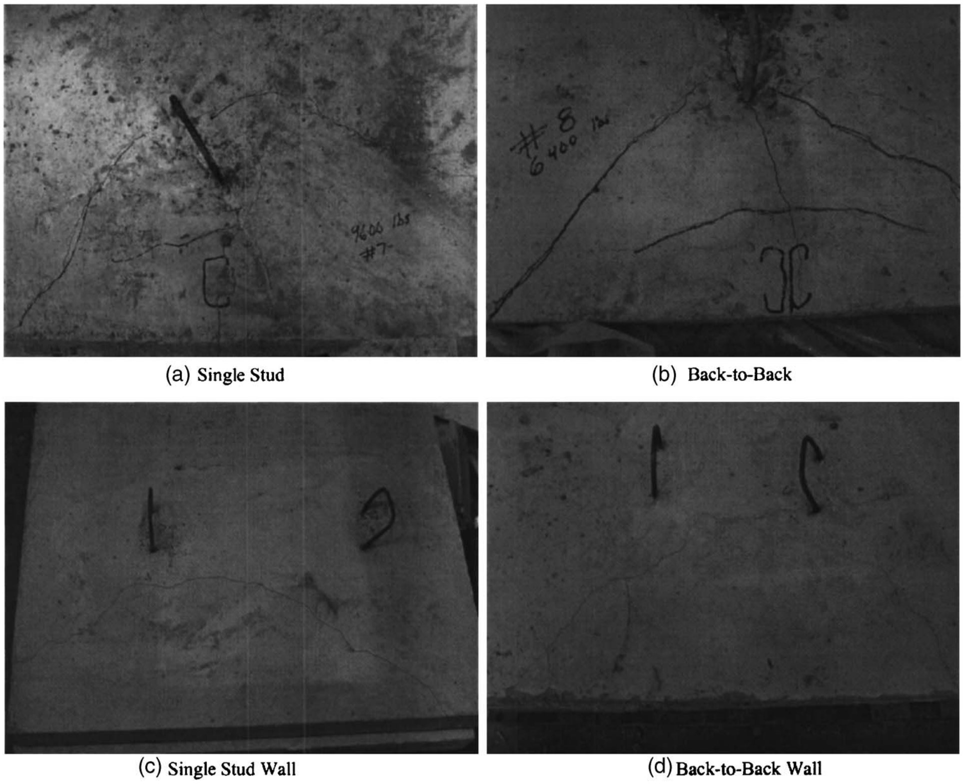

Fig. 7. Typical failure modes of test slabs

Test Results and Discussions

Evaluation of the Current Specifications and Recommendations

ACI 318-02

The maximum bearing load on the concrete is defined in ACI 318-02 (Sec. 10-17) as shown in Eq. (1)

$$P_u = \phi(0.85 f'_c A_1) \qquad (1)$$

where f'_c = concrete strength; ϕ = strength reduction factor taken as 0.7; and A_1 = area of the contact surface as defined in Fig. 8.

When the supporting surface is wider on all sides than the loaded area, the maximum bearing load may be taken as

$$P_u = \phi(0.85 f'_c A_1)\sqrt{\frac{A_2}{A_1}} \quad \left(\text{with } \sqrt{\frac{A_2}{A_1}} \leqslant 2.0\right) \qquad (2)$$

where A_2 = area of the lower base of a right pyramid or cone formed by extending lines out from the sides of the bearing area at a slope of 2 horizontal to 1 vertical to the point where the first such line intersects an edge as defined in Fig. 8.

The 2:1 rule used to define A_2 does not imply that the load spreads at this slope; it is merely an empirical relationship derived based on the test performed by Hawkins (1968), in which unreinforced concrete blocks were loaded through a rigid plate. However, the 2:1 rule may not be applicable to a cold-formed steel wall, since the contact surface area of cold-formed steel stud section is significantly smaller than that of the rigid plate.

In Fig. 9, the bearing loads of all the tested specimens calculated by ACI 318-02 were compared to the test results. In these calculations, the strength reduction factor ϕ was not used for comparison purpose and the cross-sectional areas of the studs were used for the contact surface area A_1; in fact, it is not clearly stated how to determine contact surface area A_1 for the stud-track assembly in ACI 318-02. As shown in Fig. 9, the predictions of ACI 318-02 for the bearing capacity of concrete seemed to underestimate the bearing capacity of slabs on grade with a load-bearing cold-formed steel wall in all cases. The reason for the underestimation may be attributed to the underestimation of the contact surface area A_1: because of the load distribution through the track section under the stud, the contact surface area A_1 could be larger than the cross-sectional areas of the stud. Moreover, ACI 318-02 does not consider the confinement effect, showing the constant bearing capacity along the distance from the edge of the slabs while the test results showed in all cases that the bearing capacity increased gradually, probably due to the confinement effect.

Trestain's Method

The method proposed by Trestain (1991) mainly aimed to provide a method to calculate the bearing area under the bottom track of an axial load-bearing stud. The bearing area can be determined by setting the required/applied moment M_{req} at the maximum allowable concrete stress equal to the allowable moment capacity M_{all} of the track section as shown in Eqs. (3)–(5) and in Fig. 10

$$M_{req} = 0.85 f'_c X^2 / 2W_c \qquad (3)$$

$$M_{all} = ZF_y / W_b \qquad (4)$$

$$M_{req} = M_{all} \qquad (5)$$

where X = width of track assumed to cantilever beyond the face of the bearing stud, which distributes the bearing stress through the track into the concrete; W_c and W_b = safety factors taken as 2.5

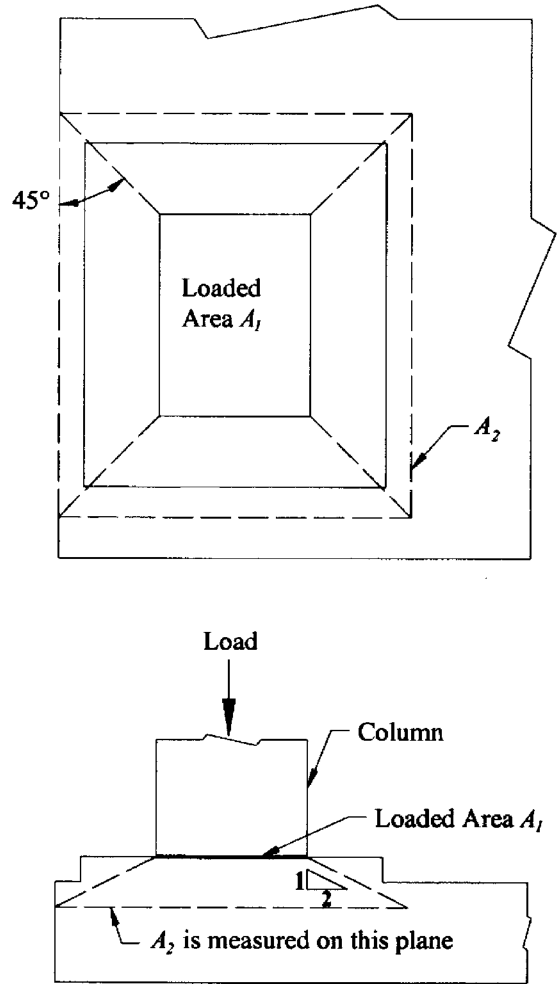

Fig. 8. Definition of loaded area A_1 and A_2

and 1.67, respectively; Z=plastic section modulus $(0.25bt_t^2)$; and F_y=yield strength of the track section.

Solving Eq. (5) for X, the following equations for the bearing area A_{brg} and the allowable bearing load P_{all} are obtained as shown in Eqs. (6)–(8):

$$X = 0.9384t_t(F_y/f_c')^{1/2} \tag{6}$$

$$A_{brg} = (F+2X)(L+X)(2) + [W-2(L+X)](t_s+2X) \tag{7}$$

$$P_{all} = A_{brg}(0.85f_c'/W_c) \tag{8}$$

Symbols used in Eq. (7) are shown in Fig. 4. In Fig. 9, the test results were compared to the predictions by Trestain (1991). In these calculations, the safety factors were not used. As shown in Fig. 9, the method proposed by Trestain significantly overestimated the bearing capacities of all the tested specimens. It could be due to the calculation of the bearing areas by Trestain's method being overestimated. The overestimation of the bearing areas might be due to the basic assumption that the allowable moment M_{all} of the track section is determined based on the plastic section modulus Z; however, it seemed that the track section would not yield until the slabs on grade failed and thus the moment applied to the track section could be considerably smaller than the allowable moment M_{all} of the track section. In addition, Trestain's method could not predict the gradual increase in bearing capacity due to the confinement effect.

Bearing Area A_{brg} (Effective Loading Area)

As described in the previous section, the accurate estimation of the bearing area is an important factor for the prediction of the bearing capacity of the slabs on grade supporting cold-formed steel wall. Thus, in an attempt to estimate the bearing area A_{brg} of the stud-track assembly, a concept of frustum was adapted as shown in Fig. 11 in this research project. The stud-track assembly systems transfer the load applied to the stud through its bottom track to the concrete bearing surface. In the frustum concept, the load applied to stud is spread to the bottom of the track at a slope of 1:1, which was determined by the elastic finite-element analysis as shown in Fig. 12. The finite-element model used a plane stress field model and consisted of the stud and track sections with a unit thickness. A unit stress was applied to the stud to determine the stress distribution through the track section. As a result, it was found that the length of the bottom of the track that was stressed was slightly larger than three times the track thickness. Thus, the slope was conservatively taken as 1:1 for practical purpose to determine the bearing area. The bearing area A_{brg} can be then determined by the following equations:

For a single stud

$$A_{brg} = (W+2F+2L)(t_s+2t_t) \tag{9}$$

For back-to-back

$$A_{brg} = 2(W(t_s+t_t) + (2F+2L)(t_s+2t_t)) \tag{10}$$

Configuration Factor

In order to investigate the effect of the different configurations of the stud-track assembly on the bearing strength, the bearing strengths of all the tested specimens were normalized with respect to their concrete strengths as shown in Fig. 13. The bearing strengths f_{brg} of the specimens were calculated by dividing the bearing load P_{test} obtained from the test by the bearing area A_{brg} calculated by Eqs. (9) and (10) as shown in Eq. (11)

$$f_{brg} = P_{test}/A_{brg} \tag{11}$$

As shown in Fig. 13, the differences between single stud and single-stud wall and between back-to-back and back-to-back wall were not significant. However, it was clearly shown that the normalized bearing strengths of back-to-back and back-to-back wall configurations were significantly smaller than those of single stud and single-stud wall configurations. The average normalized bearing strength of single stud and single-stud wall configurations was 0.974 while that of back-to-back and back-to-back wall configurations was 0.670 as shown in Fig. 13. In an attempt to incorporate this variation in the design, a configuration factor ξ was introduced as shown in Table 2. The values of the configuration factor ξ were determined using the average normalized bearing strengths shown in Fig. 13.

Confinement Effect on the Bearing Strength

The normalized bearing strengths of all tested specimens at various distances from the edge of the slab were divided by the normalized bearing strength of the specimen tested at the edge in order to investigate the effect of wall location with respect to edge of slab. As shown in Fig. 14, the normalized bearing strength increased gradually as the wall is placed farther from the edge of the slab. This phenomenon is attributed to the confinement effect of surrounding concrete as previously stated by many researchers

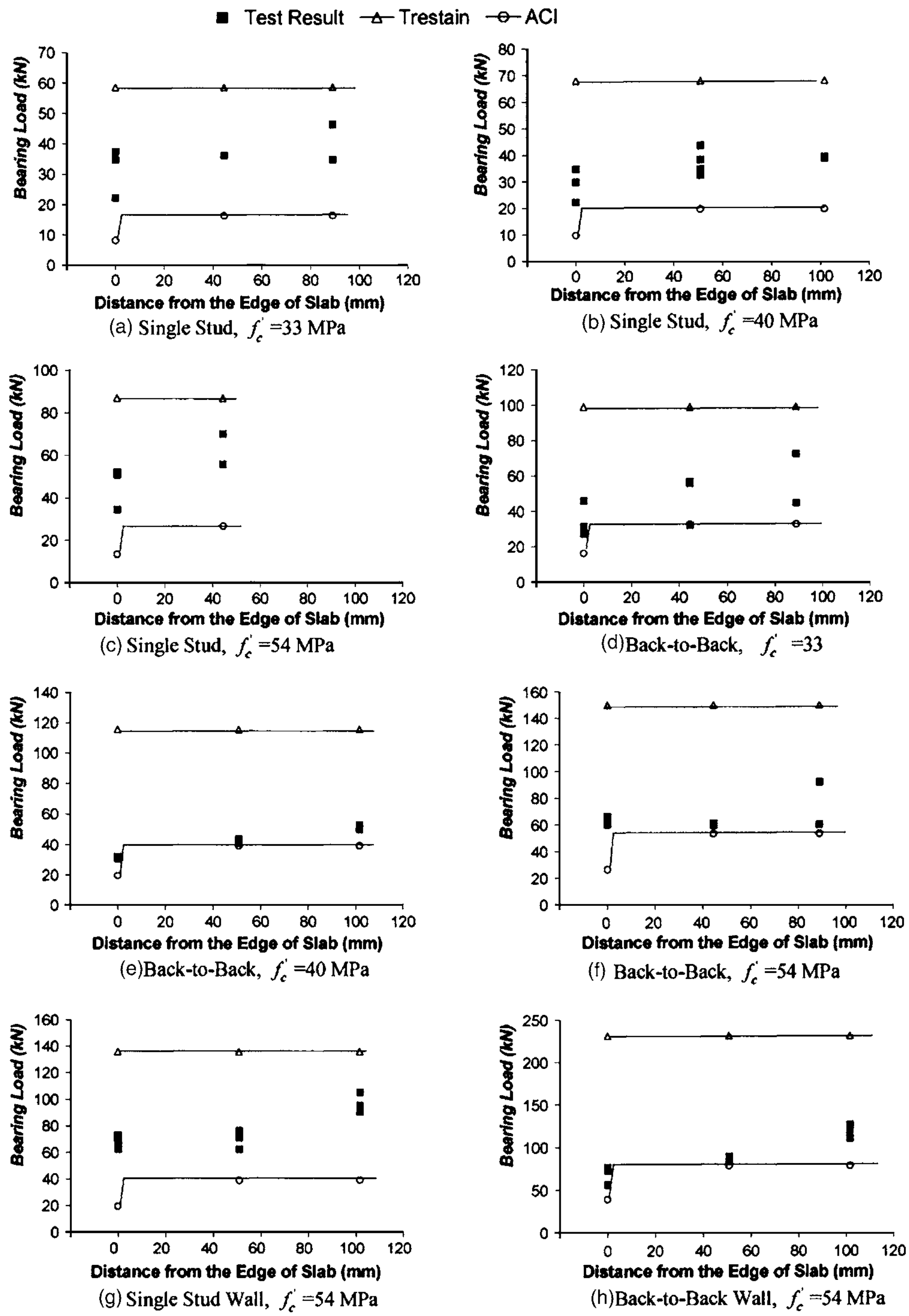

Fig. 9. Comparisons of test results and predictions by Trestain (1991) and ACI 318-02 (strength reduction factors are not used in the calculations)

(Ahmed et al. 1998; Hawkins 1968). In addition, the increases in normalized bearing strength were observed in every type of stud-track configuration and showed similar trends as illustrated in Fig. 14. This effect is expressed by the following equation:

$$P_e/P_0 = 0.5\frac{e}{t} + 1.0 \tag{12}$$

where P_e/P_0=increase rate of normalized bearing strength; e=distance from the edge of the slab; and t=thickness of the slab. It was the intention of Eq. (12) to show, based on the test results of this research project, that there may be a simple relationship between bearing strength and the distance from the edge of the

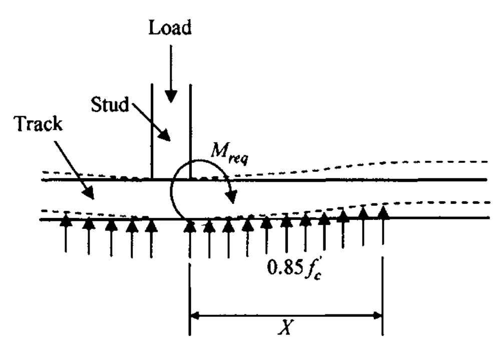

Fig. 10. Determination of X in Peyton's method

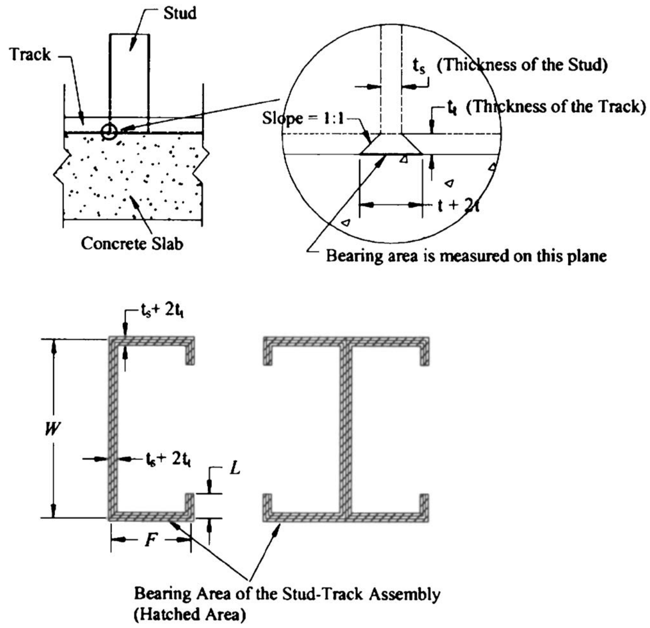

Fig. 11. Application of the frustum concept to determine the bearing area A_{brg}

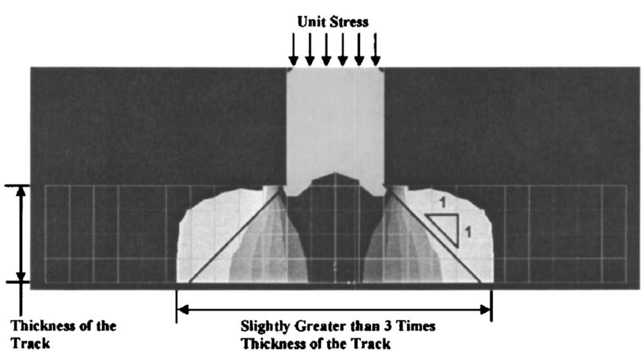

Fig. 12. Stress distribution through track section based on finite-element model analysis

Table 2. Configuration Factor ξ

Configuration factor	Single stud and single-stud wall	Back-to-back and back-to-back wall
ξ	0.974	0.670

slabs; as the distance increases, the confinement effect increases. The thickness was not a variable of this study. In order to reflect the thickness effect on the confinement, additional research work is necessary, which is the next step of our research plan.

Design Guidelines

Based on the previous discussion of the test results and analytical approach, an equation was proposed that can be used to estimate the bearing capacity P_{brg} of slabs on grade supporting the load-bearing cold-formed steel stud-track assembly, as follows:

$$P_{brg} = \phi A_{brg} 0.85 f'_c \xi \left(0.5\frac{e}{t} + 1.0\right) \tag{13}$$

where A_{brg}=bearing area calculated by Eqs. (9) and (10); ξ=configuration factor as shown in Table 2; e=distance from the edge of the slabs; and t=thickness of the slabs; e/t in Eq. (13) is limited to 1.0.

In Fig. 15, the bearing capacities P_{brg} of all the tested specimens calculated by Eq. (13) are compared to the test results P_{test}. As shown in Fig. 15, the proposed Eq. (13) is in reasonably good agreement with the test results. This equation could make it possible for design engineers to estimate the bearing capacity of slabs on grade supporting cold-formed steel wall systems that are used in residential and commercial constructions.

Conclusions

From the experimental study on the bearing capacity of slabs on grade supporting load-bearing cold-formed steel walls, the following conclusions were drawn:

1. According to the comparisons with the test results, it was found that ACI 318-02 underestimates the bearing capacity while the method proposed by Trestain (1991) overestimates

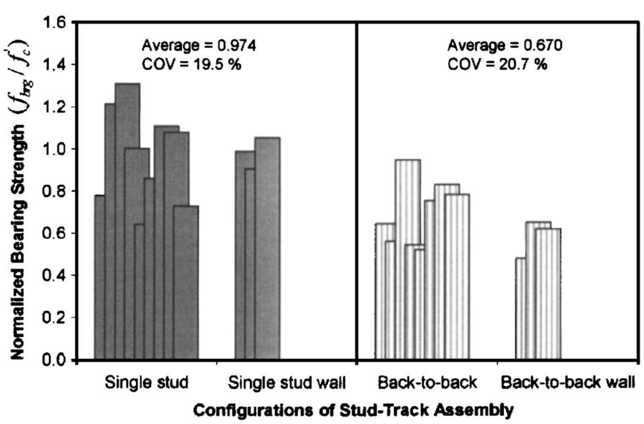

Fig. 13. Normalized bearing strength of specimens tested at the edge of the slabs

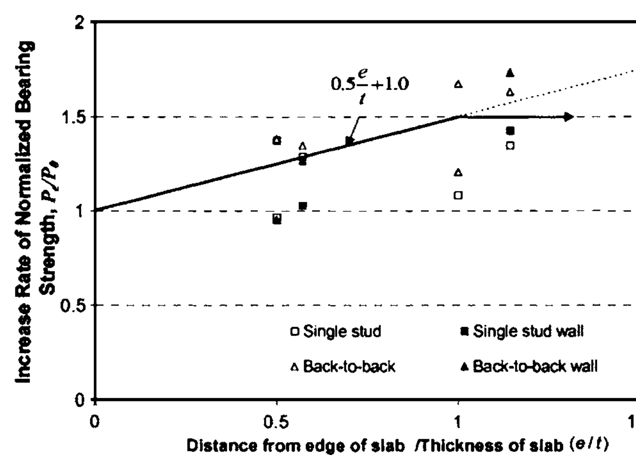

Fig. 14. Increase rate of normalized bearing strength at various distances from the edge of the slabs

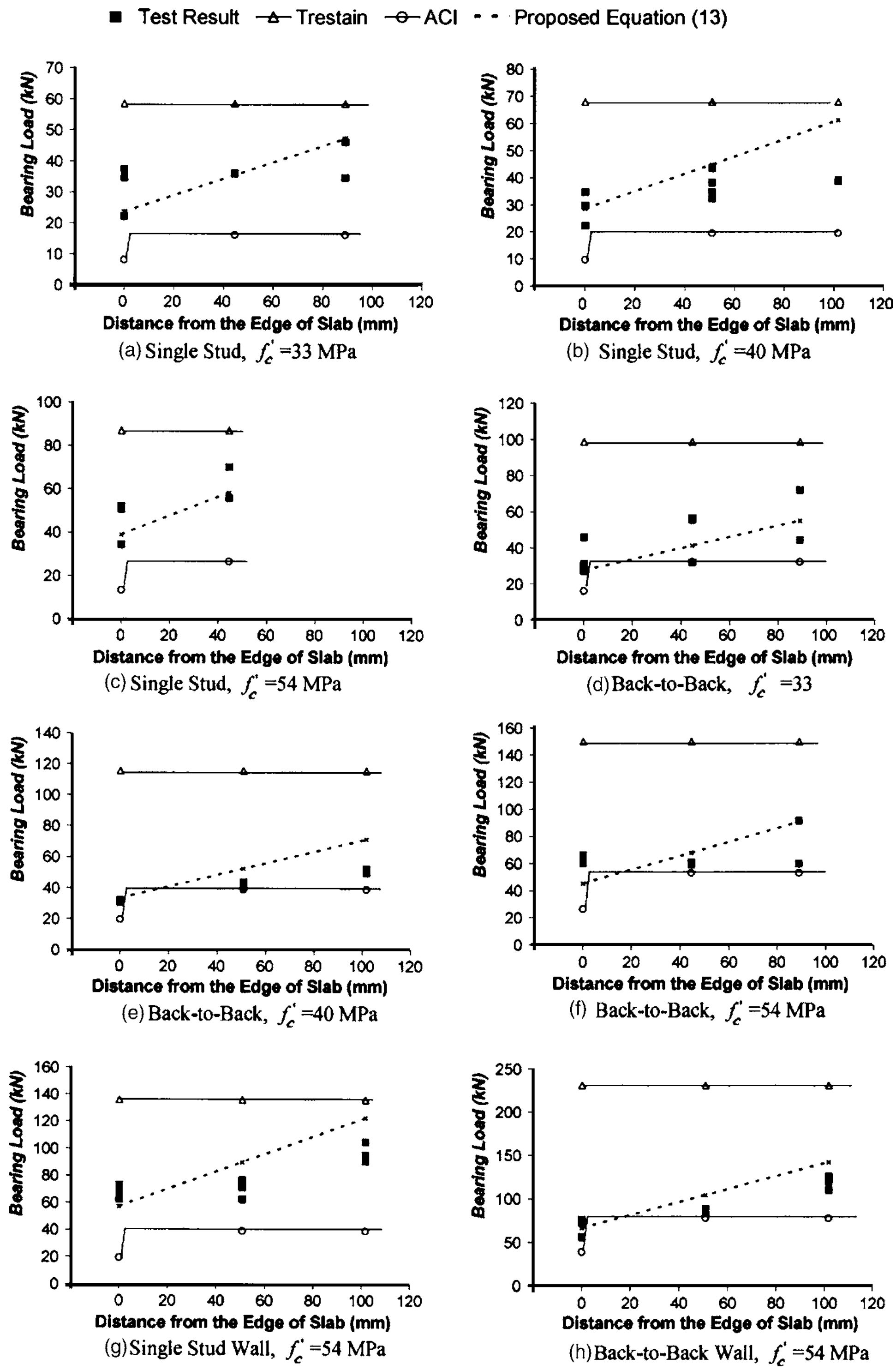

Fig. 15. Comparisons of test results and predictions by the proposed Eq. (13) (strength reduction factors are not used in the calculations)

the bearing capacity. In addition, neither of the methods could predict the increase in the bearing strength as a function of the location of wall away from the edge of slabs.

2. The inaccuracy of ACI 318-02 and the method by Trestain (1991) was due to the inappropriate estimation of the bearing area. Thus, a new method to define the bearing area is proposed based on a finite-element analysis with calibration using test results.
3. Test results showed that the configurations affected the bearing strength; back-to-back and back-to-back wall configurations exhibited a smaller bearing strength than single stud and single-stud wall configurations. In order to reflect this in the design, a configuration factor ξ was introduced.
4. Test results also showed that the bearing strength increased as the distance from the edge of the slabs increased due to the two-dimensional confinement effect of the surrounding concrete, and the rate of the increase could be expressed by a linear relationship up to a distance equal to the thickness of the slab.
5. Based on the test results, an equation was proposed to predict

the bearing strength, which utilized the proposed method to estimate the bearing area, configuration factor ξ, and the linear relationship for the confinement effect.

6. The predictions of the proposed equations were compared with test results and were shown to provide better prediction than the current ACI 318-02 and the method proposed by Trestain (1991).

Acknowledgments

Some of the experimental work was carried out by S. Phillips, P. F. Findlay, and K. Orf as a part of their undergraduate research study. Their contribution to this study is gratefully acknowledged.

Notation

The following symbols are used in this paper:

A_1 = loaded area of the contact surface as defined in Fig. 7;
A_2 = area determined by the frustum concept of ACI 318-02 as defined in Fig. 7;
A_{brg} = bearing area;
e = distance from the edge of the slab;
F = flange length of stud and track;
F_y = yield strength of the track section;
f_{brg} = bearing strengths of the specimens;
f'_c = concrete strength;
L = lip length of stud and track;
M_{all} = allowable moment capacity of the track section;
M_{req} = required/applied moment at the maximum allowable concrete stress;
P_{all} = allowable bearing load;
P_{brg} = bearing capacity of slabs on grade;
P_{test} = bearing load obtained from the tests;
t = thickness of slabs;
t_s = thickness of stud;
t_t = thickness of track;
W = web length of stud and track;
W_b = safety factors taken as 1.67;
W_c = safety factors taken as 2.5;
X = width of track assumed to cantilever beyond the face of the bearing stud;
Z = plastic section modulus;
ξ = configuration factor; and
ϕ = strength reduction factor.

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