Newsletter for the
Light Gauge Steel Engineers Association
Fall 2002

INSIDE

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— 2003 —

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Convention: Mar 25-30
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What's New on Cold-Formed Steel Framed Shear Walls in Building Codes

by H. W. Martin P.E., Director Construction Codes and Standards American Iron and Steel Institute (AISI)

Before the end of the year, the 2003 International Building Code (IBC) and International Residential Code (IRC) will be finalized, as will the 2003 Building Construction and Safety Code (BCSC), the new building code published by the National Fire Protection Association (NFPA). Although many in the industry were disappointed to see that there would not be a single national model code, there may be some consolation in the fact that both the IBC and the Building Construction and Safety Code codes will treat the design of cold-formed steel framed buildings in much the same manner.

Each of these new building codes has adopted the following recently completed AISI standards:

- North American Specification for the Design of Cold-Formed Steel Structural Members (NASPEC)
- Standard for Cold -Formed Steel Framing—Header Design
- Standard for Cold-Formed Steel Framing—General Provisions
- Standard for Cold-Formed Steel Framing—Truss Design
- Standard for Cold-Formed Steel Framing—Prescriptive Method for One- and Two-Family Dwellings

The last three standards were also adopted by the International Residential Code. The NASPEC and Header Design Standard were not introduced into the IRC since they are design documents and not prescriptive standards.

Both the NFPA and ICC (International Code Council) building codes reference the same standards for steel design. Since there are no standards presently for steel shear wall design, it was necessary to include similar provisions within the body of both building codes. Hopefully, this will not be the case in the next edition of the codes, since the AISI Committee on Framing Standards is currently developing a design standard on lateral design that will replace this information. The target date for the adoption of the new AISI standard is the 2006 edition of these building codes. Persons wishing information on the development of this new standard may contact the AISI COFS (Com-

Continued on page 2

New Standards Update Engineering Notes

The American Iron and Steel Institute's Committee on Framing Standards (AISI COFS) has recently published four standards, approved by the American National Standards Institute (ANSI). They are:

- Standard for Cold-Formed Steel Framing – General Provisions (AISI/COFS/GP 2001),
- Standard for Cold-Formed Steel Framing – Prescriptive Method for One and Two Family Dwellings (AISI/COFS/PM 2001),
- Standard for Cold-Formed Steel Framing – Header Design (AISI/COFS/HEADER 2001),
- Standard for Cold-Formed Steel Framing – Truss Design (AISI/COFS/TRUSS 2001).

The General Provisions document serves as a base for the others, and is referenced by each. It includes definitions, nomenclature, references, and basic information on members, connection, and design as it applies specifically to cold-formed steel framed construction. This differs from the AISI Specification and the new North American Specification, since the focus of the General Provisions docu-

Continued on page 7

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Shear Walls

Continued from page 1

mittee on Framing Standards) Secretariat for more information (kbielat@steel.org). Engineers who have used the shear wall provisions in previous editions of the IBC or one of the three model codes will be pleasantly surprised with the enhancements in the 2003 building codes.

Following are frequently asked questions regarding the shear wall provisions in building codes.

Will the new codes permit me to use perforated shear walls?

Yes. The codes will now permit the use of the perforated wall design approach for steel-framed walls in a manner similar to that permitted for wood-framed walls. Unlike the traditional shear wall design, where hold-down devices are required at the end of each wall segment, in perforated walls hold downs are only required at the ends of each wall.

Can I use Gypsum Board for Seismic Resistance?

The new codes treat the use of Gypsum Board for seismic resistance as a brittle material. The design forces are determined using a very low R value (2.5) and limit the height of the systems in high seismic areas. The table for Gypsum Board attached to steel framing may now be used for both wind and seismic applications.

Why are the shear values in the steel chapter so much higher than the wood chapter?

The shear values for steel framing are intended to be equally applicable to the use of both LRFD and ASD. Since LRFD and ASD are included within the NASPEC and its predecessor specification, it was felt that it would be best to follow the same format for the design of cold-formed steel shear walls. The shear values for steel framing are thus presented as nominal values - that is, they must be multiplied by a resistance factor (ϕ) [LRFD] or reduced by a factor of safety (Ω) [ASD]. Once the nominal shear values in the steel chapter have been reduced by a factor of safety, they are similar to the ASD values in the wood chapters.

Can I use narrow shear walls?

Yes. The new codes permit the use of steel-framed shear walls with height (h) to width (w) ratios of 4:1 for some assemblies, for both wind and seismic applications. It should be pointed out, however, that when walls are permitted to have a h/w ratio of 4:1, the nominal shear value must be reduced by a factor of 2(w/h). Testing has shown that narrow (4:1) walls can produce the same strengths as wider (2:1) walls but at a reduced stiffness. To account for this reduced stiffness, the design values must be reduced accordingly. This concept was introduced into the 2000 NEHRP Recommended Provisions for Seismic Regulations for New Buildings (FEMA 368/March 2001).

Why is there an overstrength factor Ω_o?

In seismic design, the code design forces are not intended to be elastic design forces. This is because the seismic forces are really fictitious forces, since the expected forces have been reduced by an R factor to determine the design seismic forces. In the design event, it is expected that the system will be operating in the inelastic range of performance and the response of the system will be controlled by the capacity of the weakest element. Thus, it is important that elements (for example, boundary members and anchorage) attaching the primary lateral load resisting element (sheathing or x-bracing) to other parts of the structure or foundation have sufficient strength to facilitate development of ductile response of the primary lateral load resisting elements. As such, the overstrength factor is intended to assure that the anchors and end studs not fail in a brittle manner before the primary lateral load resisting element. It is important to remember that when checking member strengths against this amplified load, it is the member nominal strength that needs to meet or exceed the demand: not the allowable or design strength. Using the latter strengths will result in members being larger than required by code.

Can I use double-sided shear walls for steel-framed walls?

Previous codes did not permit this since the systems for steel were all based upon tested values. However, since the 2000 NEHRP Provisions included requirements for the use of additive shear values for wood walls, it was felt that doing so for steel would be appropriate. Thus, the new codes include provisions to allow a designer to add shear values for identical materials on both sides of a wall. Please check with the codes for the specific rules. Care must be taken in de-

Continued on page 5

R-Factor Project Validation Test Plan and Preliminary Results (Part 1)

by Jim Wilcoski, U.S. Army, Corps of Engineers

In October 2000, the Construction Engineering Research Laboratory (CERL) began a project to characterize the inelastic response of structural systems. Ductile behavior is critical to good structural performance of buildings in earthquakes. Current building code provisions recognize degrees of assumed ductile behavior through the use of a seismic response modification factor, R. Seismic loads used to design the vast majority of buildings (linear static or dynamic design) are inversely proportional to this factor. Values for these factors vary from a low of 1.5 (ordinary plain masonry shear walls) to a maximum of 8.0 (special steel moment frames). These values are intended to represent the degree of ductility, overstrength, redundancy and energy dissipation capacity of the structural system. These factors have a tremendous impact on the design of buildings, yet there is no rational basis for the establishment of these values. As was stated in FEMA 303, section 5.2 “the R values, contained in the current Provisions, are largely based on engineering judgment of the performance of the various materials and systems in past earthquakes. The values of R must be chosen and used with careful judgment.”

However, the static cyclic testing does not account for dynamic effects that will be experienced in real earthquakes. The hysteretic load versus deflection plots of cold-formed steel shear panels are severely pinched, because the main panel lateral load resisting elements are thin diagonal straps that only offer resistance under tensile load. After a deformation cycle that causes strap yielding, the panel will have almost no resistance until deformations have cycled in the opposite direction to amplitudes that the opposite straps become taut. While the straps are slack, the structure above the panel can develop significant velocity, and the straps can snap when they become taut again, causing large accelerations, and shock loading of the joints. The strap connections to the columns must not fail, the columns must not buckle, or the anchors of the columns must not fail, as any of these failures could be brittle and are not represented by the ductile hysteretic behavior defined in the laboratory. Other structural systems that have pinched hysteretic envelopes may have similar issues. The static cyclic tests also do not represent the large P-delta-related overturning moments that could result at large deformations of multi-story building frames. Therefore shaketable testing of a full-scale model is needed to evaluate the effectiveness of the non-linear analysis in representing the dynamic response of structures at large deformations. This validation testing is needed to evaluate the ability to define the deformation demand and capacity of the lateral load resisting system.

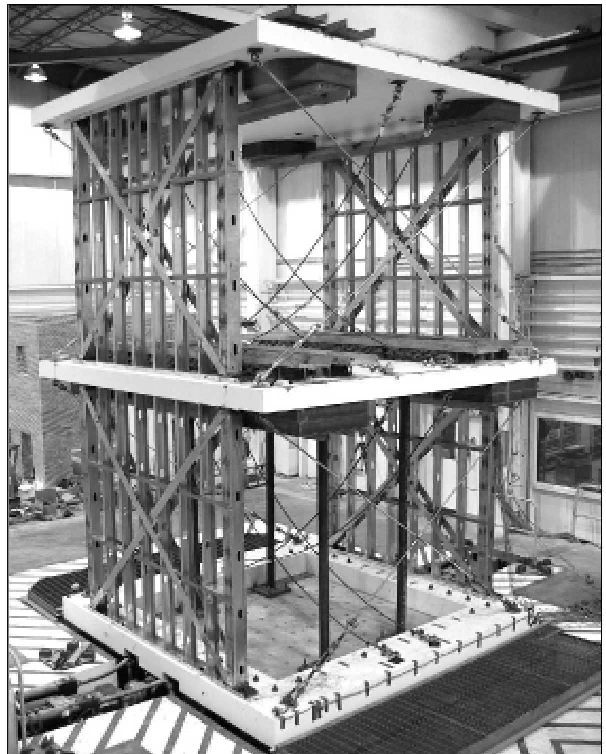

Figure 1 cold-formed steel shaketable model

Shaketable Model Configuration

Figure 1 shows a photograph of the cold-formed steel shaketable model assembled on the CERL Triaxial Earthquake and Shock Simulator (TESS shaketable). The model is full-scale, consisting of two framing lines of 2-story, cold-formed steel shear panels. Supplemental weight has been added above and below the floor slabs.

This model was shaken with uniaxial motions in the in-plane direction relative to the walls. The model consists of two identical two-story, one bay-wide frames, that are separated from each other by 154 in. on center in the out-of-plane direction. The second-story frame is identical to the first story, though the loads on first story are greater, so that significant non-linear response should occur on the first story only, where it can be more easily observed during the test. A heavy reinforced concrete slab diaphragm was installed at the top of each floor level. The concrete slabs were 8 in. thick and 14 ft 6 in. square. The slabs are very stiff, representing a beam at the top of the wall panels, plus they add weight to the model. Each slab weighs 21,000 lbs. In a typical building shear panels might be installed in one of every 5 or 10 bays of a building. Therefore, additional mass was added to the model that might come from other bays of a typical building. All the available steel plate and lead weights at CERL that could be easily installed on the model were evenly distributed on the two slabs. This included approximately 24,000 lbs of steel plates and 40,000 lbs of lead, plus channels to hold them in place. The effective model weight at the first floor was 57,500 lb, and the effective weight at the second floor was 57,600 lbs. The heavy slabs were intended to prevent flexural bending or in-plane rotation of the floor diaphragms, so that the single bay frame would provide similar frame response as a multiple bay building frame. The columns at the exterior edges of the frame were very stiff axially and had moment connections to the slabs with through bolts to the base beam below and slabs above.

Half-inch threaded rod braces were installed out-of-plane to prevent unwanted out-of-plane model response. In the unlikely event that the panel diagonal straps fail in a brittle manner, loose cables were installed in the in-plane direction to “catch” the slabs.

Continued on page 7

The Light Gauge Steel Engineers Association needs you and your experience. Please mail or fax your opinions, questions, and design details that are relevant to the cold-formed steel industry (fax to Dean Peyton at (253) 941-9939. Upon editorial review, your submission may be printed in the Technical Exchange Section of this Newsletter.

Strength of Floor Joists with Offset Loading on Bearing Stiffeners

by Steven R. Fox, PhD, P.Eng., General Manager
Canadian Sheet Steel Building Institute

The AISI Committee on Framing Standards (AISI COFS) has recently published the updated "Standard for Cold-Formed Steel Framing – General Provisions" [AISI/COFS/GP 2001]. This document provides requirements for construction with cold-formed steel framing that are common to prescriptive and engineered design. One of the requirements in the General Provisions Standard calls for "in-line" framing unless a structural load distribution member is included. In-line framing means that the "joist, rafter, truss and structural wall stud shall be aligned so that the centerline (mid-width) is within 3/4-inch (19 mm) of the centerline (mid width) of the load bearing members beneath (see Figure C1-1).

It is common practice in cold-formed steel construction to include bearing stiffeners in floor joists at each bearing location or point of concentrated load. An extensive study [Fox and Schuster 2002] of the behavior of bearing stiffeners has been carried out, and new design provisions are being proposed for inclusion in the North American Specification for the Design of Cold-Formed Steel Structural Members [AISI 2002]. The basic design equations for bearing stiffeners, however, do not recognize the influence of a 3/4-inch offset in the load path through the assembly. A pilot research project was undertaken at the University of Waterloo [Black et. al. 2002] to look at the effects on the behavior of the assembly by offsetting the load path.

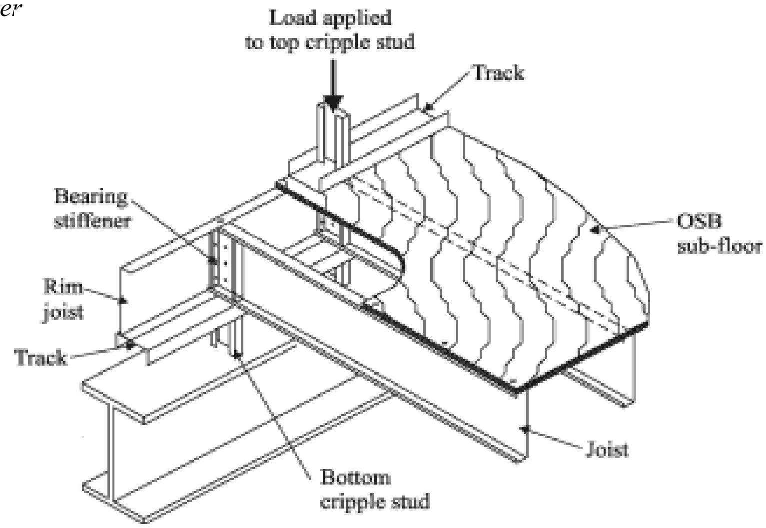

Figure 1: Cut-Away Sketch of Test Set-Up

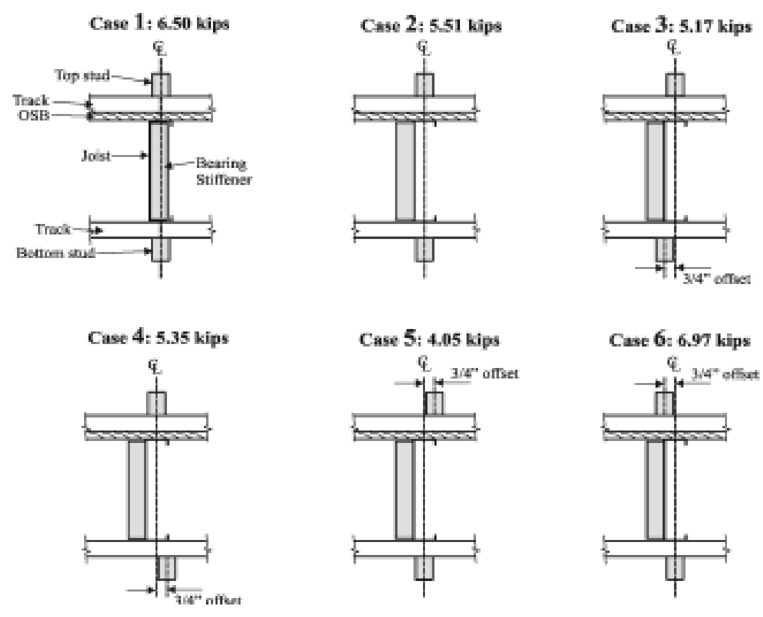

Figure 2: Offset Load Cases

Test Configurations

The test assemblies in the project were constructed to simulate actual floor assemblies. Each specimen consisted of four floor joists in a four-foot square assembly. The cut-away drawing in Figure 1 illustrates the details. The load was applied through a short cripple stud to one end of one of the floor joists.

The floor joists and rim joists were 8 inches deep with an actual base steel thickness of 50 mils. The bearing stiffeners were 3-5/8 inch stud sections with a measured thickness of 34 mils. The sub-floor was 5/8 inch OSB. The wall framing track sections were 3-5/8 inch wide with a nominal thickness of 33 mils. The cripple studs were 3-5/8 inches wide with a nominal thickness of 105 mils.

The provision of the 3/4-inch offset between the centerlines of the various components creates a number of possible configurations: (a) the top loadbearing stud can be offset on either side of the joist; (b) the bottom loadbearing stud can be offset on either side of the joist; (c) the bearing stiffener can be attached on either side of the joist web. The behavior of the assembly will be influenced by the combination of stud offset and stiffener location. The test configurations considered are illustrated in Figure 2. These configurations were selected to cover the more common variations.

Test Results

The tested capacities of each assembly are also shown in Figure 2. These values are the average of two tests of identical specimens.

Case 1 is the "base-line case" and provides a load path that is in a direct line through the studs and bearing stiffener.

Continued on page 5

Shear Walls

Continued from page 2

sign, especially seismic design, to assure that the chord members and anchors are able to accommodate the additional forces developed by including materials on both sides.

In seismic design, the demands on the end studs and anchors can be estimated by multiplying the nominal capacity of the sheathing by the height of the wall. This seems counterintuitive, but it works: the effects of the lengths of the wall cancel out. Thus if one wanted to design a double-sided shear wall of 15/32-in. plywood, 8 ft. tall with screws spaced 2 in. on center, (nominal shear value is 2190 plf for a single side), the end stud demand would be 2*(2190)*(8) = 35,040 lbs. That means that one should be designing end studs and anchors for a nominal strength of 35 kips! Use caution, because the demands add up in a hurry.

Why is the thickness of the stud limited in seismic applications?

Initially, the shear walls were tested using 33mil and 43 mil studs with sheathing attached with No. 8 screws. Concerns were expressed that thicker materials would cause the screws to fracture rather than have the failure in the sheathing material. To assure that only 33 and 43 mil studs were used, the codes require the stud's thickness to be limited to 43 mil. Recent testing has indicated that when larger screws are used, thicker studs are appropriate. It is expected that this will be covered in the new AISI standard.

Hopefully this gives you a snapshot of what's coming in the new codes. You may want to review specific provisions, and ask your local building department for permission to use them before they are adopted. ❒

Strength of Floor Joists

Continued from page 4

Case 2 has the stiffener attached to the back of the joist, which results in a reduction in capacity of approximately 18% compared to Case 1. As a matter of interest, the capacity of this assembly calculated according to the bearing stiffener design provisions being proposed for the North American Specification (NASPEC) would be 4.72 kips. This shows that the effect of the rim joist and sub-floor can potentially increase the capacity of the assembly by up to 35%.

Cases 3 and 4 investigated varying the offset of the bottom loadbearing stud. There was no significant difference in strength with variations in the bottom stud offset (i.e. compare Cases 2, 3 and 4). It was originally expected that the failure of the assembly would be initiated at this bottom stud location since there is no load distribution element, similar to the sub-floor under the top stud. However, in all of the tests, the failure initiated at the top of the joist. This may be attributed to distribution of load through the rim joist and sub-floor to the other bottom cripple studs supporting the assembly.

Cases 5 and 6 investigated varying the offset of the top loadbearing stud. Case 5 clearly shows the effect of the offset of the top stud from the bearing stiffener. In all other tests except Case 5, the failure mode was web crippling of the joist followed by local buckling of the stiffener. In Case 5, the failure was punching of the top loadbearing stud through the sub-floor. This occurred even though there was a track between the sub-floor and the stud. Case 6 shows that if the top stud is located over the stiffener, the results are comparable to Case 1 with the stiffener between the joist flanges.

Conclusions

This article has described a pilot study investigating the significance of the 3/4-inch offset allowed with in-line framing. It was found that the significant variable affecting the capacity of the assembly was the offset between the upper loadbearing stud and the bearing stiffener. A reduction in strength of as much as 40% was observed. Based on the calculated capacity of 4.72 kips, one of the test values was less than computed nominal values (Case 5). Based on this preliminary information, designers and builders may want to avoid the double offset condition (Case 5) that creates this reduction. This research report is available from the Web site of the Canadian Cold Formed Steel Research Group at www.civil.uwaterloo.ca/ccfsrg. ❒

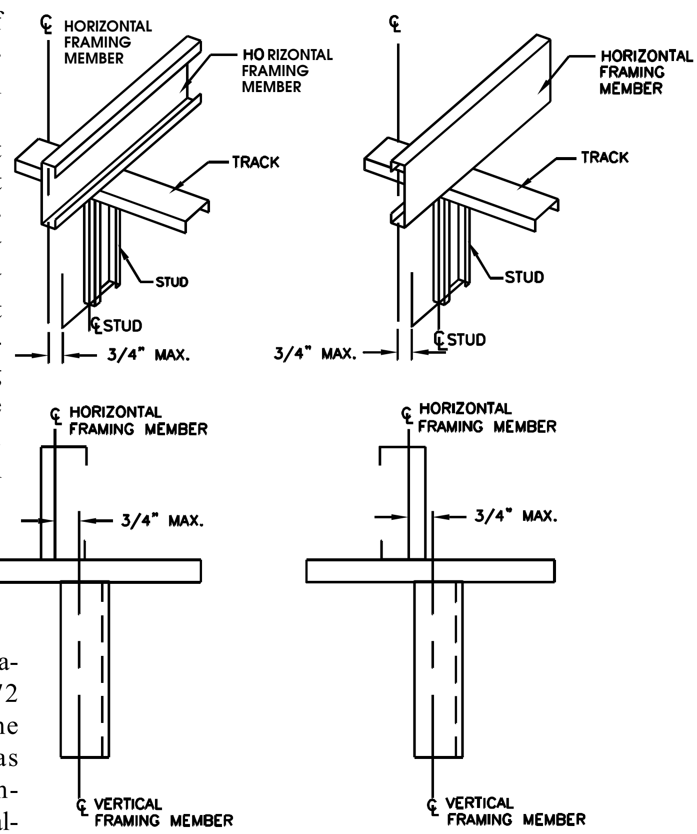

Fig. C1-1 In-Line Framing. *Each joist, rafter truss and structural wall stud shall be aligned vertically so that the centerline (mid-width) is within 3/4 inch (19 mm) of the centerline (mid-width) of the load bearing member beneath per Figure C1-1. The 3/4 inch (19 mm) maximum alignment tolerance is not required when a structural load distribution member is specified in accordance with an approved design or a recognized design standard.*

Anchorage Design Example for Light Gauge Cold-Formed Wood Structural Panel Sheathed Shear Walls Per the 1997 Uniform Building Code

by Steven E. Pryor, S.E., Simpson Strong-Tie Company

The following design example discusses how to properly select holdowns to resist overturning in shear walls framed with steel studs and sheathed with wood structural panels in accordance with the 1997 UBC Chapter 22, Division VIII (hereafter referred to as "the Code"). The requirements for wind and seismic differ, so both are included.

Design Philosophy

Designing for wind is more straightforward than designing for seismic, because the design wind forces are indicative of the real expected maximum design wind forces, whereas the design seismic forces are not. The design seismic forces are some fraction of the expected real forces for the design level event, and the Code provides the overstrength factor, Ω_o, to help the designer estimate this difference. The Code's seismic provisions are based on the assumption of some [form of] inelastic response from the structure, and as the R factor increases so does the assumed inelastic response.

In order for the designated lateral force resisting system to be able to achieve the expected response, it is important to identify the members and connections of the system that are controlled primarily by strength. For example, a double C-stud post used as a boundary element in a shear wall needs to be stronger than the overturning demand caused by the attached sheathing. This will allow the sheathing to develop its expected inelastic performance as required by the system. The same is true for other perimeter members and connections. This becomes more critical in higher seismic zones. As a result, Section 2220 [of the 1997 UBC] has additional requirements for both members and connections used to resist seismic demand in seismic zones 3 and 4.

Design Example

Given:

1) 4 ft. x 8 ft. shear wall with an allowable stress design (ASD) level shear demand for both wind and seismic equal to 1500 lb
2) Negligible gravity loads on the wall and no net wind uplift on the wall
3) 7/16 in. rated sheathing (OSB) one side attached with No. 8 x 5/8 self drilling screws per Tables 22-VIII-A and C
4) 43-mil framing
5) Ω_o = 2.8 (system overstrength factor)
6) Seismic Zone 3

Required:

1) Screw spacing and holdown selection for both wind and seismic forces.

Screw Spacing:

1) PLF demand = 1500 lb / 4 ft. = 375 PLF (same demand assumed for both wind and seismic).
2) Wind (reference section 2219.3, safety factor Ω =3*)
   a. From Table 22-VIII-A, choose screw spacing = 4 in. o.c., good for 1410 plf / (Ω = 3) = 470 plf
   b. 470 plf > 375 plf OK
   ** Safety factor used in the International Building Code is 2.5*
3) Seismic (reference section 2219.3, safety factor Ω =2.5)
   a. From Table 22-VIII-C choose screw spacing = 3 in. o.c., good for 1275* plf / (W = 2.5) = 510 plf
   b. 510 plf > 375 plf OK
   ** UBC values shown here are based on 33 mil (20 gauge) studs.*

Holdown Selection:

1) Wind:
   a. Holdown Demand = 1500 lb x (8 ft. / 4 ft.) = 3000 lb (uplift)
   b. If design load was already reduced by 0.75 in the load combination, or if the set of load combinations used for design does not allow a 1/3 stress increase, then:
      i. Increase design uplift by 1/3, thus design uplift = 1.33 * 3000 lb = 4000 lb
      ii. Check manufacturer's 133 values (i.e. values holdown values that already include 1/3 stress increase): Use a holdown good for a minimum of 4000 lb and check concrete requirements.
   c. If the load combinations used for design do allow for a 1/3 stress increase, then:
      i. Check manufacturer's 133 values for 3000 lb uplift
      ii. Use a holdown good for a minimum of 3000 lb and check concrete requirements.
2) Seismic:
   a. Holdown Demand
      i. Seismic uplift demand = 3000 lb at the ASD level.
      ii. Per 2220.2, the strength level uplift shall be increased by the Ω_o factor. Increase the ASD uplift by 1.4 to convert to the strength level. Design uplift = 3000 x 1.4 x 2.8 = 11760 lb. This value need not exceed what the wall is capable of transferring to the holdown = wall nominal strength (plf) x wall height = 1275 x 8 = 10200 lb. Therefore, design uplift = 10200 lb.
   b. If the holdown manufacturer does not offer strength (LRFD) level holdown resistance (typically the case), convert the manufacturer's 133 value to a LRFD value by multiplying the manufacturer's value by 1.7/1.33 (= 1.28)—though not explicitly stated in the Code, this approach is consistent with Sections 1630.8.2.1 and 2213.4.2.
   c. Use a holdown that has a 133 value good for a minimum of 7970 lb (= 10200/1.28 lb) and check the concrete requirements.

Conclusion:

a. Seismic design controls.
b. Seismic requirement is almost twice that of wind.
c. It is evident that even if the seismic demand is less than the wind demand, in some cases seismic demand may still control design of the screw spacing, boundary members, and anchorage. ❐

R-Factor Project

Continued from page 3

Two 5/8 in. diameter cables were installed in each direction at each floor level and their lengths were such that they became taut and carried load at 12 in lateral deflection at each floor level. In the unlikely event that the model columns buckled initiating collapse, four-4 in. diameter double-extra-strong pipe columns were installed below the first floor slabs. The column heights are such that the plates at the tops of the columns are two inches below the first floor slabs.

Shaketable Model Design

The shear panels installed in this model at both floor levels are actual full-scale wall panels, designed for a base shear of 15.6 kips per panel or 32.2 kips for both frames (see Table 1). The straps were welded to the columns at their tops and bottoms. The connections and the columns themselves, plus the anchors to the base beam and floor slabs above were all designed following the guidance in the Corps TI 809-07. These components are designed for the maximum overstrength of the straps as required by this Technical Instruction (TI), so that the model would behave in the desired ductile manner.

The guidance in Corps TI 809-07 requires the use of an R factor of 4. However, in order to test the non-linear demand and capacity at extreme deformations, the cold-formed steel shear panels were undersized for the loads applied to them, based on an R of 4.

The model behavior was predicted using the same analytical procedures for defining model drift demand and capacity that are used to calculate recommended R factors. This analysis was conducted on two-dimensional frames using DRAIN 2DX non-linear finite element analysis software. Validation or verification of this analytical approach requires reasonable agreement. The predicted behavior was based on modeling the shaketable model frames using the actual material properties of the diagonal straps and columns determined in the coupon tests. The analysis used the estimated weight of the model described earlier under model configuration. The columns were assumed fully fixed, though in reality the column anchors will have some finite rotational stiffness. ❒

Note to readers: The conclusion of the article, including a report on test results, will be published in the next issue of the LGSEA Newsletter.

Panel Location	Panel Width W (in.)	Panel Height H (in.)	Strap Faces n_s (#)	Strap Width b_s (in.)	Strap Design Thickness (ga.)	t_s (in.)	Strap Initial Lat. Stiffness k_s (k / in.)	Yield Stress of Strap F_{sy} (ksi)	Strap Lat. Yield Capacity Q_{sy} (kips)	Design Shear Strength $\phi_t Q_{sy}$ (kips)	Lat Defl at Strap Yielding δ_{sy} (in.)	Applied Story Shear $V_x+Q_{s\,i}$ (kips)	Elastic Lateral Deflection δ_{xe} (in.)	Defl Amp Factor C_d	Import Factor I	Design Story Drifts $\Delta=\delta_x$ (in.)	Stability Coeff. θ	Allow Story Drifts Δ_a (in.)
2nd Story	118	118	2	4	16	0.0548	38	53	16.4	15.608	0.431	14.396	0.378	3.5	1.0	1.32	0.0064	2.36
First Story	118	118	2	4	16	0.0548	38	53	16.4	15.608	0.431	21.581	0.567	3.5	1.0	1.32	0.0128	2.36

Table 1 Cold-Formed Steel Diagonal Strap Design for Shaketable Model

General Provisions

Continued from page 1

ment is on framing design only: utilizing “C” shaped studs, tracks, and accessories. It therefore includes only requirements unique to construction framing. Examples include the permissible gap below a stud seated in a track, isolation of plumbing and electrical utilities, and the acceptable percentage of stripped screws in a shear connection.

Because of the enforceable nature of these provisions, it is an excellent document to be referenced in architectural specifications (typically section 05400) and structural general notes. Similarly, the truss and header documents should be referenced when those assemblies are used as part of the framing package. For more information or to order copies of these documents, go to the LGSEA Web site. LGSEA members are qualified for discount pricing. ❒

CCFSS Announces 16th International Specialty Conference on Cold-Formed Steel

The 16th International Specialty Conference on Cold-Formed Steel Structures is scheduled to take place October 17-18, 2002 in Orlando, Florida. This conference will be presented by the Department of Civil Engineering of the University of Missouri-Rolla and the Wei-Wen Yu Center for Cold-Formed Steel Structures. The event is designed to bring together leading scientists, researchers, educators, and engineers who have been engaged in the field of research and design of cold-formed steel structures for discussion of recent research findings and design considerations.

As with previous specialty conferences, which have been held since 1971, this conference will include the presentation of technical papers and the publication of a volume of conference proceedings. A total of 56 papers are scheduled for presentation in several fields of interest including, Element Behavior, Flexural Members, Web Crippling of Beams, Compression Members, Rack Structures, Wall Studs, Building Systems, Materials and Connections. Of particular interest to the steel framing industry are seven papers pertaining to wall stud behavior and design. For a brief abstract of the papers see the “Newsletters and Technical Bulletins” page on the Center’s website at www.umr.edu/~ccfss.

The conference will be held at the Wyndham Orlando Resort, which is conveniently located on Orlando’s International Drive, near such area attractions as Walt Disney World, Universal Studios, and Sea World. Advance registration is requested. For more information, contact the Center by email at ccfss@umr.edu, or phone (573) 341-4471. ❒

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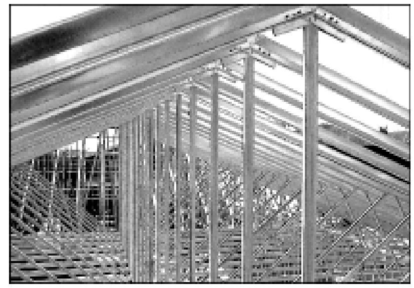

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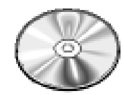

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