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UNIFIED FACILITIES CRITERIA (UFC)

COLD-FORMED LOAD BEARING STEEL SYSTEMS AND MASONRY VENEER/STEEL STUD WALLS

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COLD-FORMED LOAD BEARING STEEL SYSTEMS AND MASONRY VENEER/STEEL STUD WALLS

U.S. ARMY CORPS OF ENGINEERS (Preparing Activity)

NAVAL FACILITIES ENGINEERING COMMAND

AIR FORCE CIVIL ENGINEER SUPPORT AGENCY

Record of Changes (changes are indicated by **1**\ ... /**1**/)

Change No.	Date	Location

This UFC supersedes UFC 3-310-04A, dated 1 March 2005. The format of this UFC does not conform to UFC 1-300-01; however, the format will be adjusted to conform at the next technical revision. The body of this UFC is the previous TI 809-07, dated 30 November 1998

FOREWORD

\1\
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UNIFIED FACILITIES CRITERIA (UFC)

DESIGN: COLD-FORMED LOAD BEARING STEEL SYSTEMS AND MASONRY VENEER/STEEL STUD WALLS

The text of this UFC is the previous TI 809-07, dated 30 November 1998.

TI 809-07

US Army Corps
of Engineers

TI 809-07
30 November 1998

Technical Instructions

Design of Cold-Formed Loadbearing Steel Systems and Masonry Veneer / Steel Stud Walls

Headquarters
U.S. Army Corps of Engineers
Engineering and Construction Division
Directorate of Military Programs
Washington, DC 20314-1000

TECHNICAL INSTRUCTIONS

Design of Cold-Formed Load Bearing Steel Systems and Masonry Veneer / Steel Stud Walls

Record of Changes (changes indicated \1\…/1/)
No. Date Location

Supersedes: ETL 1110-3-439 Masonry Veneer/Steel Stud Walls (Nonload-bearing Construction)

FOREWORD

These technical instructions (TI) provide design and construction criteria and apply to all U.S. Army Corps of Engineers (USACE) commands having military construction responsibilities. TI will be used for all Army projects and for projects executed for other military services or work for other customers where appropriate.

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FOR THE COMMANDER:

Dwight A. Beranek, P.E.
Chief, Engineering and Construction Division
Directorate of Military Programs

DESIGN OF COLD-FORMED LOADBEARING STEEL SYSTEMS

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CHAPTER 1

INTRODUCTION

1. PURPOSE AND SCOPE. This document provides design guidance on the use of cold-formed steel systems for both load-bearing (Chapter 2) and nonload-bearing (Chapter 4) applications. Also, criteria are provided to use load-bearing systems in shear wall (Chapter 3) applications. The nonload-bearing application guidance in Chapter 4 also includes provisions for the masonry wythe and moisture protection of complete masonry veneer/steel stud curtain wall systems.

2. APPLICABILITY. These instructions are applicable to all elements responsible for the design of military construction. Exceptions to this criteria will require Corps of Engineers Headquarters (CEMP-ET) approval.

3. REFERENCES. Appendix A contains a list of references used in these instructions.

4. PARTNERING EFFORT. This document is the result of partnering between industry and Government with emphasis on Green Building Technology. Designers should require materials, products and innovative construction methods and techniques which are environmentally sensitive, take advantage of recycling and conserve natural resources. Funding for this effort came from the Green Building Program.

5. DESIGN CONCERNS UNIQUE TO COLD-FORMING.

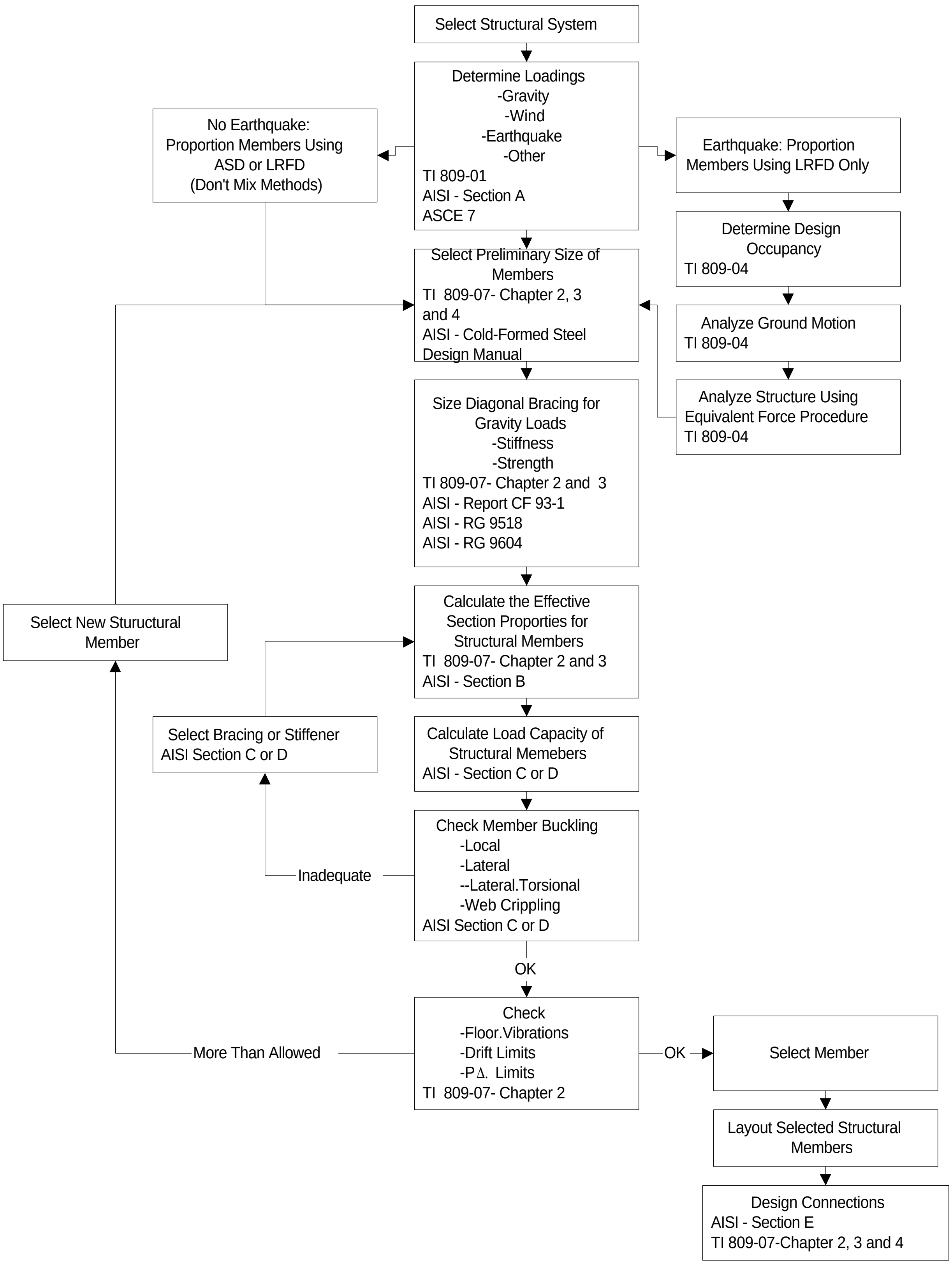
a. General. The AISI Specification is applicable to sheet and strip steels with thicknesses of 6.35 mm (¼ in) or less, but steel plates and bars up to 25.4 mm (1 in) can successfully be used as structural shapes. Designers working with cold-formed steel products will account for several unique conditions not normally found in AISC steel designs as outlined below:

- Effective section properties are based on the design stress of the loading condition being analyzed,
- Lateral-torsional buckling is a special buckling condition unique to the cold-formed design of compression members,
- Local buckling of section elements, and the lateral buckling of members is different than typically found in AISC steel design,
- Connections can be assembled using welds, screws, or bolts. Crimping is not allowed as well as powder driven pins. However, powder driven pins can be used to attach wall tracks to concrete floors, foundations, and steel superstructures,
- A Deign Process for Load-bearing Cold-Formed Steel Systems Flowchart is shown as Figure 1-1.

b. Seismic Design The seismic design guidance provided in Chapter 3 will only be used for Performance Objective 1A (Life Safety Performance Objective Level defined in TI 809-04) for all seismic ground motion levels and enhanced Performance Objectives (2A, 2B and 3B) for Seismic Design Categories A and B. The definition of performance objectives and seismic design categories are provided in TI 809-04, Seismic Design for Buildings.

Seismic design with cold-formed steel has two inherent problems with the material itself. The first is light gauge thickness of the cold-formed steel materials and the second is the material strength variability. The objective of seismic design guidance is to ensure ductile building system performance in the large seismic event and elastic response in the small event or wind loading. Ductile building performance requires that selected ductile components yield, but continue to carry loads and absorb energy through significant plastic response. At the same time potentially brittle failure modes, such as column buckling or connection failure must be prevented. The design challenge for cold-formed steel is to ensure that building components, and in particular shear panel components, be proportioned

Figure 1-1: Design Process for Loadbearing Cold-Formed Steel Systems



relative to each other and detailed to ensure the ductile response. In this guidance, this is accomplished by ensuring that the diagonal straps yield and respond plastically through significant displacement, without risk of damage to brittle connections or column buckling.

Seismic design guidance is provided on three levels:

- Tabular data for prototype shear panels in terms of the maximum story shear and maximum and minimum gravity load. These terms are defined in Chapter 3 and the shear panel configurations and data are provided in Appendix E.
- Detailed guidance for shear loads using shear panels with diagonal straps as primary lateral load resisting element. This guidance is provided in Chapter 3, with background guidance taken from other sources in Appendix C and an example problem illustrating the guidance in Appendix D. The spreadsheet, http://owww.cecer.army.mil/techreports/wilcfsxl.post.pdf program used in the example problem is available as a design tool for shear panel design.
- A test procedure and the acceptance criteria for other shear panel configurations is provided in Appendix F.

6. USES OF COLD-FORMED STEEL.

a. General. Cold-formed steel systems have been used in industry and within the government for many decades. The primary areas of usage include: standing seam roof systems, doors, roof and floor joists, decking and floor systems, ventilation and ceiling systems, interior wall partitions and exterior fascia, metal buildings, lighting poles, guardrail, and corrugated steel pipe. These TI provide guidance necessary for designers to develop loadbearing steel systems.

b. Steel Framing Systems. These systems can be used in: wall, floor, and roof trusses of low rise offices, single family homes, and multi-family housing structures. Cold-formed systems should be galvanized for the local environmental conditions, and be pre-punched for routing utility services through walls. A rubber or plastic grommet must be provided in each pre-punched hole that utilities are passing through to prevent corrosion between dissimilar metals in the wall stud cavity.

7. COLD-FORMED SUPPLIERS. Cold-formed manufacturers perform the following services for the commercial customers: provide framing details, develop design information on the structural capacity of their members, help specify the job's materials, and deliver precut materials. Government designers should specify the design properties of each structural member; studs, track, heads, jambs, jack studs, etc. These properties include member depth, width and design thickness for each section.

8. RESPONSIBILITIES

a. Designers. Cold-formed steel framing involves the use of engineered products outlined in an overall engineered system. Cold-formed steel framing is ideally suited for structural applications such as curtain wall framing, load-bearing wall framing, floor framing, rigid frames, trusses, and roof rafter framing.

(1) Selecting Member Strength. Designers are responsible for specifying which products are to be used as well as which standards the products must meet. The products must meet all of the design requirements which normal engineering practice requires. Axial load capacity of compression members, flexure, torsion, shear, and web crippling in beams, and combined stresses in columns, etc. must be examined.

- Designers are to specify all materials by nominal uncoated thickness, effective section properties, and member loads. Loads should include moment and shear diagrams, and axial load with eccentricity.
- All connections (member-to-member and member-to-structure) are to be examined carefully taking into account all relevant physical strengths and properties as well as proper transfer of all loads to the supporting structure.

- Design selected walls to provide frame stability and lateral load resistance.
- Engineered methods such as diaphragm shear walls or diagonal steel strapping are used to provide frame stability and transfer lateral loads through the structure into the foundation. Provide additional studs as required to resist the vertical component of the loads from the diagonal bracing.
- Wall bridging is designed to provide resistance to minor axis bending and rotation of studs.
- Diaphragm rated components can be substituted for bridging; however, they must be installed prior to loading of the wall. If components are installed on one side of the wall only, then the flanges on the other side of the studs must be bridged with suitable bridging. Bridging can be removed or left in place when diaphragm rated components are installed.
- Provide for structure movement as indicated and necessary by design or code requirements.

(2) Member Specification. The primary specification for the design of the individual components is the AISI Specification for the Design of Cold-Formed Steel Structural Members. Other considerations, which the designer specifies, include:

- Member or Stud size, spacing, and depth.
- Deflection criteria, maximum spacing, minimum gage, wind loads.
- Studs are spaced to suit the design requirements and limitations of collateral materials.
- Allow for additional studs at panel intersections, corners, doors windows, control joints, etc.

b. Manufacturers. Lightweight steel framing involves the use of engineered products in an engineered system. Manufacturers provide technical product information, including physical and structural properties, and manufacturing standards that the designer can use to specify the appropriate structural products.

- Samples. Samples are representative pieces of all framing component parts and accessories;
- Certification. Certification is a statement from the manufacturer certifying that the materials conform to the appropriate requirements as outlined in the contract documents;

c. Contractor. The Contractor is responsible for providing the finished structure in strict compliance with the contract documents and cold-formed steel shop drawings. Included in the contractor's responsibilities are the following:

- Shop Drawings of details and attachments to adjoining work.
- Drawings showing plans, elevations, sections, and details.
- shop coatings
- steel thickness
- size, location, spacing of fasteners for attaching framing to itself
- details of attachment to the structure
- accessories and their installation
- and critical installation procedures

d. Contractor Quality Control. The Contractors Quality Control (CQC) Manager inspects Corps of Engineers projects. This effort outlines the contractor's inspection document requirements. The following Quality Assurance Documents are to be maintained by the CQC Manager:

- The manufacturer and product delivery dates,
- the contractor's staff and qualifications,
- compliance to testing standards for members, erection tolerances, and fasteners,

- submittal registers, erection drawings, production drawings,
- designer of record approvals, and CQC inspection reports,
- designer of record notes to the field on handling, storage, installation and special inspections by design engineering staff during construction,
- designer of record limitations and permissible deviations,
- qualifications of field welders,
- process for getting designer of record approval for applying attachments to metal stud systems,
- final inspection and approval of framing systems (this could be accomplished through the local building code group).

9. MATERIALS. The marking standard used in the following material does not have a metric equivalent. Materials marked to the following standard are for field acceptance of the delivered materials. Designers are to use the nominal uncoated material thickness as shown on the drawings and within the specification.

a. Markings. The Metal Lath / Steel Framing Association (ML/SFA), Metal Stud Manufacturers Association (MSMA), the AISI Residential Advisory Group (RAG), and the Prescriptive Standard for Steel Framing, accepted by the Council of American Building Officials (CABO), have adopted a standard method of marking all cold-formed steel members. The markings will be on the web of the section and will be repeated throughout the length at a maximum of 1219 mm (48 inches) on center. There is no defined size or method of marking, but the markings must be legible and easily read. The product marking will include the following information:

(1) Manufacturer identification. Manufacturer’s Name, Company Logo or emblem will be displayed to clearly identify the product manufacturer.

(2) Minimum delivered uncoated steel thickness. Material thickness without coatings or galvanizing is represented in mils of the decimal thickness value for example 0.84 mm (33 mil);

(3) Protective coating designator. The galvanizing coat designator will indicate the amount level of protection being provided such as: G-40-EQ, G-60-EQ, or G-90-EQ, where EQ is an equivalent protective coating to the designated galvanized coating.

(4) Minimum yield strength. Material strength in kips per square inch (ksi)

b. Nomenclature.

(1) Standard Nomenclature for Manufacturer’s Catalogs:

(a) The first set of numbers will represent the depth of the section to two decimal places without the use of a decimal point.

(b) A Letter following the depth of the section will define the shape of the section per the following:

“S” = CEE shaped Stud
“U” = Channel shaped Stud
“T” = Track
“F” = Furring Channel

(c) The second set of numbers will represent the flange width to two decimal places without the use of a decimal point.

(d) The last set of numbers will be the minimum delivered thickness in mils and be preceded by a dash (-).

Example: 362S162-33 = 92 mm (3-5/8") CEE Stud with a 41 mm (l-5/8") Flange – 0.84 mm (33 mils) Thickness

(2) Standard Minimum Delivered Uncoated Metal Thicknesses are shown in Table 1-1:

Table 1-1: Standard Minimum Delivered Uncoated Metal Thickness

Nominal / Design Thickness			Minimum / Delivered Thickness		
Gage	Soft Metric	Decimal	Soft Metric	Decimal	Mils
26	0.437 mm	0.0172"	0.414 mm	0.0163"	16
25	0.478 mm	0.0188"	0.455 mm	0.0179"	18
22	0.719 mm	0.0283"	0.686 mm	0.0270"	27
20	0.879 mm	0.0346"	0.836 mm	0.0329"	33
18	1.146 mm	0.0451"	1.087 mm	0.0428"	43
16	1.438 mm	0.0566"	1.367 mm	0.0538"	54
14	1.811 mm	0.0713"	1.720 mm	0.0677"	68
12	2.583 mm	0.1017"	2.454 mm	0.0966"	97
10	3.150 mm	0.1240"	2.997 mm	0.1180"	118

From ICBO: Acceptance Criteria for Steel Studs, Joists, and Tracks, AC46, April 1998.

(3) Standard Flange and Return Lips for CEE Studs:
0.46 mm & 0.69 mm (18 & 27 Mil) minimum thickness = 32 mm (1-1/4") Flange with 4.8 mm (3/16") Lip.
Any Thickness = 35 mm (1-3/8") Flange with 9.5 mm (3/8") Lip.
Any Thickness = 41 mm (1-5/8") Flange with 13 mm (1/2") Lip.
Any Thickness = 51 mm (2") Flange with 16 mm (5/8") Lip.
Any Thickness = 64 mm (2-1/2") Flange with 16 mm (5/8") Lip.

(4) Standard Track Flange Sizes: 25 mm (1"), 32 mm (1-1/4"), 38 mm (1-1/2") and 51 mm (2")

(5) Standard "U" Channels: Web x Leg x Minimum Thickness
29 mm (3/4") x 13 mm (l/2") x 1.37 mm (54 Mil)
38 mm (1-1/2") x 13 mm (1/2") x 1.37 mm (54 Mil)
51 mm (2") x 13 mm (1/2") x 1.37 mm (54 Mil)

(6) Standard Hat Furring Channels: Depth x Minimum Thickness
22 mm (7/8") x 0.46, 0.69,0.84, 1.09 mm (18, 27, 33, and 43 Mil)
38 mm (1-1/2") x 0.46, 0.69,0.84, 1.09 mm (18, 27, 33, 43, and 54 Mil)

c. Strain Hardening.

(1) General. The cold working of steel stresses it beyond its elastic limit and leaves a residual strain. After the cold working, the steel has an increased yield stress. This process of raising the yield stress is known as cold working. The amount of strain hardening is relative to the amount of cold working. In normal roll forming, the cold working is nominal, the steel remains ductile and it makes a modest improvement in the allowable stress of the section. Extensive cold working of steel can cause it to loose its ductility and become brittle, but normally, this condition is not associated with common roll formed products.

(2) Cold Reducing and Rerolling of Steel. Steel mills have the capability to cold reduce the steel sheet to meet the specified thickness before shipment to the manufacturer. Some manufacturers can reroll the steel sheet to again reduce the sheet thickness. Within the AISI Specification this is known as Other Steel. This practice is allowed by the AISI Specification if the rerolled steel meets the ductility criteria of the AISI Section A3.3. Therefore, the manufacturer is strain hardening the material.

Depending on the degree of rerolling that occurs, the material will have a higher steel yield strength, less ductility, and a more rounded stress-strain curve above the yield point of the steel than would be seen in the virgin steel sheet materials. Design guidance that is provided in chapter 3 recognizes this material property variability. Diagonal bracing materials are to be ASTM A653 steel without rerolling, and will be a Category I submittal for approval by the designer of record.

d. Damaged Materials.

(1) General. Steel framing materials can be rejected for the following reasons: physical damage (dents, cuts, twists, buckles); corrosion; length; metal thickness; yield stress; protective coating or forming. Physical damage and corrosion are easily identified. Length variations and forming problems are relatively easy to identify with standard measuring devices. The metal thickness must be checked with a micrometer and one must know what the required thickness should be. The yield strength and protective coating can only be checked through testing.

(2) Tolerances and Coatings. The allowable physical tolerances and standard protective coatings for steel framing products can be found in current ASTM Standards. ASTM C-645 covers tolerances and standard protective coatings for nonstructural partition framing and ASTM C-955 covers the same information for structural framing. The yield strength for structural framing is normally specified in the project specifications or on the drawings. Special protective coating requirements are also specified in the project specifications.

e. Fire Resistance Rating.

(1) General. Fire rating of assemblies denotes a length of time that a given assembly will resist fire penetration under controlled laboratory conditions; Table 1-2 lists many Fire Rated Assemblies. These fire-rating tests are performed in accordance with the existing consensus standards of ASTM or ANSI. A fire rating is only valid for the tested assembly. Although most materials cannot be added or changed in a fire rated assembly, stronger steel framing sections can be used. The specified depth and gauge of steel framing in a tested assembly are considered minimums. The fire rating will still apply to the assembly when steel framing members that are deeper and/or heavier than the specified members, are used.

(2) Fire Tests. There are numerous fire rated assemblies for interior walls, exterior walls, floors, ceilings, and roofs, incorporating steel framing. Various laboratories throughout the country have performed fire rating tests of loadbearing steel stud walls. Currently, within the AISI Residential Advisory Group (RAG), a task group has been assigned to compile a listing of all available fire rated assemblies, using steel framing. The following sources list many fire rated assemblies that can be used if cold-formed design.

- Fire-Resistance Ratings of Load-Bearing Steel Stud Walls (AISI, Publication Z-4)
- Fire Design Manual (Gypsum Association)

Table 1-2: Fire Rated Assemblies
The following table depicts various fire rated assemblies incorporating steel framing components.

Test Reference	Fire Rating	Type of Assembly	Agency	Components
FM 24676.4 FC 224	2 HR	Floor/Ceiling	FM 1975	-64 mm (2 ½") concrete (note B) -14 mm (9/16") 0.36 mm (28 GA) deck and mesh -184 mm (7 ¼") X 1.15 mm (18GA) joists, 610 mm (24") OC -2 layers 16 mm (5/8") G.W.B. ceiling
FM 29135 FC 245	1 HR	Floor/Ceiling	FM 1977	-51 mm (2") concrete -41 mm (1 5/16"), 0.56 mm (24 GA) deck -152 mm (6") X 1.15 mm (18 GA) joists, 610 mm (24") OC -1 layer 13 mm (½") G.W.B. ceiling
L 524	1 HR	Floor/Ceiling	UL 1988	-Min 184 mm (7 ¼") X 1.15 mm (18 GA), steel stud, 610 mm (24") OC -Use any of the floor systems indicated in the UL test
P 511	1 HR	Roof/Ceiling	UL 1988	-Min 184 mm (7 ¼") X 1.15 mm (18 GA), Steel Joist, C Shape, 51 mm (2") Flange Minimum, 610 mm (24" OC) -See test for roof/ceiling components
P 512	1 HR	Roof/Ceiling	UL 1988	-Min 184 mm (7 ¼") X 1.15 mm (18 GA), Steel Joist, C Shape, 610 mm (24") OC -See test for roof/ceiling components
U 418	¾ HR	Bearing Wall	UL 1988	See test
U 418	1 HR	Bearing Wall	UL 1988	-2 layers 13 mm (½") thick, G.W.B, one side -89 or 140 mm (3 ½" or 5 ½") X 1.15 mm (18 GA) steel stud, 610 mm (24") OC -See test for exterior component
U 418	2 HR	Bearing Wall	UL 1988	-3 layers 13 mm (½") thick, G.W.B, one side -89 or 140 mm (3 ½" or 5 ½") X 1.15 mm (18 GA) steel stud, 610 mm (24") OC -See test for exterior component
U 425	¾, 1 HR	Bearing Wall Interior	UL 1988	See test
U 425	1 ½ HR	Bearing Wall Interior	UL 1988	-2 layers 13 mm (½") thick, G.W.B, each side -89 mm (3 ½") X 0.87 mm (20 GA) steel stud, 610 mm (24" OC)
U 425	2 HR	Bearing Wall Interior	UL 1988	-3 layers 13 mm (½") thick, G.W.B -89 mm (3 ½") X 0.87 mm (20 GA) steel stud, 610 mm (24") OC
U 425	¾, 1, 1 ½ HR	Bearing Wall Exterior	UL 1988	See test
U 425	2 HR	Bearing Wall Exterior	UL 1988	-3 layers 13mm (½") thick, G.W.B, interior side -89 mm (3 ½") X 0.87 mm (20 GA) steel stud, 610 mm (24") OC -See test for exterior component
U 426	3 HR	Bearing Wall	UL 1988	-4 layers 13 mm (½") thick, G.W.B, each side -89 mm (3 ½") X 0.87 mm (20 GA) steel stud, 610 mm (24") OC
U 434	1 HR	Bearing Wall	UL 1988	-1 layer 16 mm (5/8") thick, G.W.B, interior side -89 mm (3 ½") X 0.87 mm (20 GA) steel stud, 610 mm (24") OC -22 mm (7/8") thick Portland Cement Plaster

Notes:
A: UL denotes Underwriters Laboratories, Inc., and FM denotes Factory Mutual Research Corporation.
B: Lightweight concrete measured from top flute of deck.

CHAPTER 2

COLD-FORMED STEEL DESIGN

1. INTRODUCTION. There are three design methods available to the designer: direct application of the AISI Specification, prescriptive methods, and testing of designed assemblies. The AISI specification can be used for the bulk of design calculations when selecting beams, columns, web stiffeners, and effective section properties. AISI has guides for shear walls and cold-formed steel trusses. These design guides can be used for Corps of Engineers projects, but are limited to all steel designs. The use of plywood and oriented strand board are not allowed in permanent Military design. When a designed assembly does not fall within the limits of these design guides, those assemblies must be tested. The prescriptive method that follows should only be used within its stated limits, and was created to serve the single family housing industry.

2. PRESCRIPTIVE METHODS.

a. General. One method of designing one and two family homes is to use the Prescriptive Methods for residential Cold-Formed Steel Framing, by the U.S. Department of Housing and Urban Development and the National Home Builders Association. The limitations for the prescriptive method of design are summarized in the table below and apply to residential type construction. Building loads are determined by using the ASCE 7-93 Minimum Load Assumption for Buildings.

Table 2-1. Summary of Prescriptive Methods Limitations

Web depths		Deflection Limits	
Bearing Walls	Heights	Walls	Remarks
100 mm (4")	2.4 to 3.0 m (8 to 10 ft)	L/240	Total Load
150mm (6")	2.4 to 3.0 m (8 to 10 ft)	L/360	Live Load
Joists/Rafters	Spans	Joists	
300mm (12")	2.4 to 4.9m (8 to 16 ft)	L/240	Total Load
350mm (14")	2.1 to 6.4m (7 to 21 ft)	L/480	Live Load
400mm (16")	3.9 to 7.9m (13 to 26 ft)	Rafters	
Ceiling Joists	Spans	L/180	Total Load
100mm (4")	2.4 to 4.9m (8 to 16 ft)	L/240	Live Load
150mm (6")	2.7 to 7.0m (9 to 23 ft)	Ceiling Joists	
200mm (8")	3.0 to 7.9m (10 to 26 ft)	L/240	Total Load
250mm (10")	3.7 to 8.2m (12 to 27 ft)	Headers	
300mm (12")	3.7 to 8.8m (12 to 29 ft)	L/240	Total Load
Flange 41 mm (1.625") Lip 13 mm (0.5")	Max House Width 11 m (36 ft)	L/360	Live Load

Notes:
1. Self-drilling and tapping screws: SAE J78, use a minimum edge and end distance of 3 diameters, a minimum of three exposed threads, and components in contact with each other, no gaps between connected parts. In connections that are in shear only, a minimum edge distance of 1.5 diameters in the direction of the force shall be used.

2. Dimensions in this table were originally in U.S. Customary units. Light gage steel materials are available as soft metric equivalents.

b. Material Thickness. Designers need to specify the Design or Nominal thickness of the material required. However, the minimum thickness of material that can be delivered to the job site is 95% of the design thickness specified, see Table 1-1. When using this method of specifying materials the designer will always get a material thicker than the minimum shown in the Table 1-1, and the design factor of safety, Ω and the resistance factor, ϕ accounts for this difference in thickness. Then the material should be coated with a minimum of ASTM A294, G60 galvanized coating for corrosion protection. The following is a listing of the material's design thickness used in the Prescriptive Methods code. Also, this code has developed a standard method for marking each stud. The industry identifier of 350S162-068 is read as a 39 mm (3.50 ") Stud with a 41 mm (1.62 ") flange and a material thickness of 1.81 mm (68 mils). In general studs used in this code are the same overall dimensions as their wood stud counterpart.

- Bearing Walls: 0.84, 1.09, 1.37, 1.72, 2.45 mm (33, 43, 54, 68, 97 mils)
- Nonstructural Walls: 0.46, 0.68 mm (18, 27 mils)
- Joists/Rafters: 0.84, 1.09, 1.37, 1.72, 2.45 mm (33, 43, 54, 68, 97 mils)
- Ceiling Joists: 0.84, 1.09, 1.37, 1.72, 2.45 mm (33, 43, 54, 68, 97 mils)
- Strapping: 0.84 mm (33 mils)

c. Engineered Portions of the Prescriptive Code. When using the Council of American Building Officials (CABO) Prescriptive Code (National Association of Home Builders report) there are many conditions that need to be checked by the design engineer. When using the Prescriptive method, the design engineer shall check the selection of the buildings main structural members, as this method uses building loads from ASCE 7-93. The design loads will be upgraded to the most current ASCE 7 standard. The following components are engineered portions of the prescriptive code.

- Sheathing selection and design when over 145 Km/hr (90 mph) winds and seismic zones 3 and 4
- Hybrid systems that use steel and wood
- Wall bracing, hold-downs and uplift straps for Winds loads greater than 145 Km/hr (90mph), and for Seismic Zones 3 and 4
- Pneumatically driven fasteners, powder actuated fasteners, crimping, and welding
- Overhangs, balconies, and decks with live loads greater than 1.92 KPa (40 psf)
- Floor joist splices and the design and bracing of Cold-formed Steel floor trusses
- In-line blocking every 3.7 m (12 ft) on strap braced studs
- The approval of corner framing details
- Steel strapping and 'X' bracing
- Cathedral ceilings
- Wood rafters
- Ceiling joist and rafter splices
- Steel and/or wood roof trusses and associated bracing systems
- King stud and wall stud uplift straps and end gable uplift straps
- Load carrying track members

3. COLD-FORMED STEEL FRAMING.

a. General. There are many differences when steel designers start using lightweight steel framing systems. Generally cold-formed sections are shaped and formed from flat sheets, their original mechanical properties are changed during cold-forming, there are no standard shapes, the thickness of materials are generally less then 3 mm (1/8 inch) thick, and the predominate failure mode is buckling followed by a post-buckling strength increase.

b. Design Guidance. Guidance from the American Iron and Steel Institute (AISI) applies to materials with a thickness that is less than 5 mm (3/16 inch) thick for connections, and plate up to 25 mm (1 inch) can be cold-formed. Guidance from the American Institute of Steel Construction (AISC) applies to thicker materials. When using cold-formed stainless steel, design requirements can be found in American Society of Civil Engineers (ASCE) criteria. The 1996 AISI specification covers material usage, loading combinations (ASCE 7), the structural analysis of elements, members, and assemblies, the design of connections and joints, and the testing for special cases.

(1) Section A, General Provisions. This section discusses the limits of applicability, materials, loads allowable stress design (ASD), load resistance factor design (LRFD), strength increase due to cold working, and serviceability.

(2) Section B, Elements. This section discusses dimensional limits, the effective widths of stiffened and unstiffened elements, and stiffeners. This analysis considers the flat widths and thickness of the flange, the web, and the lip, along with the effects of any intermediate stiffeners. The element analysis is used to determine the effectiveness of the element and the design stress level for that element.

(3) Section C, Members. This section goes into the calculation of section properties, the design of tension members, flexural members, concentrically loaded members, members in combined axial and bending, and cylindrical tubular members. In Figure 2-1 is shown the common section symmetries used in cold-formed steel design. Section symmetries are defined by using the relative positions of the center of gravity (CG) and shear center (SC) of the sections geometry. Sections having separate locations for the CG and SC are singly symmetric. When they are collocated at the same point they are doubly or point (Z sections of equal flange width) symmetric.

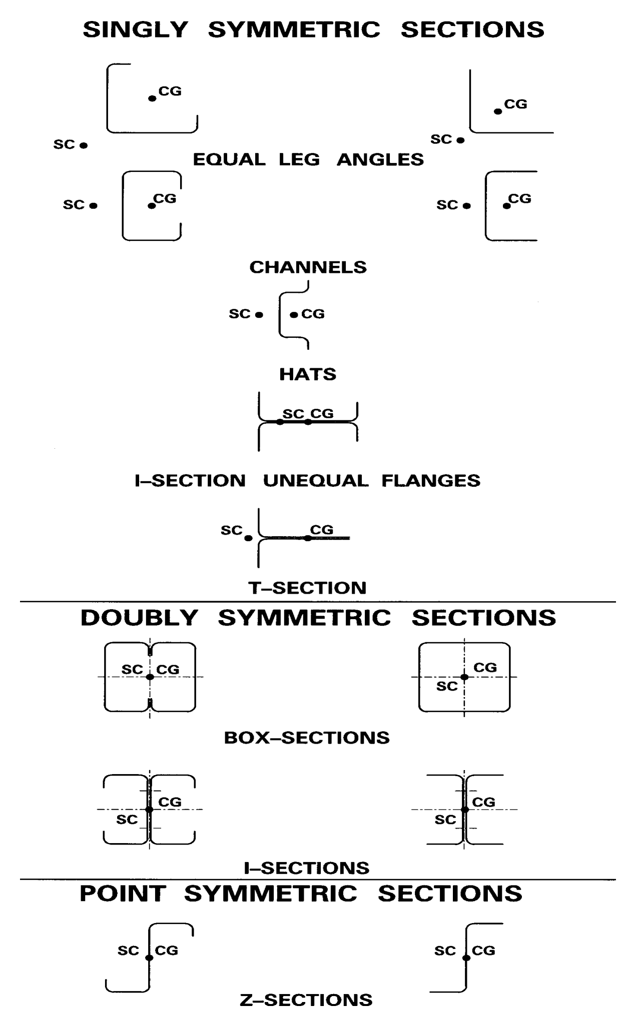


Figure 2-1 Typical Cold-Formed Section Symmetries

(4) Section D, Structural Assemblies. This section deals with structural assemblies. This section includes discussions on built-up sections, mixed systems, lateral bracing, wall stud systems, and floor, roof or wall steel diaphragm construction.

(5) Section E, Connections and Joints. This section covers the design of connections and joints, and includes welded connections, bolted connections, screw connections, shear rupture, and connection to other materials. The provisions for screw connections is new with this edition.

(6) Section F, Test for Special Cases. This section discusses testing methods and procedures for the conditions not specifically covered in the specification. This would include such tests for determining structural performance, confirming structural performance, and determining mechanical performance. Design guidance for shear walls will be performed in accordance with chapter 3 of this manual.

c. Wall Studs and Roof Trusses. Wall stud and roof framing sections are typically made from C shaped sections. These wall stud sections should always have a stiffening lip to aid in the development of the flange flat width

Table 2-2. AISI Approved Steels (Section A3.1)

Steels that are allowed and are not shown must comply with the ductility requirements of Sections A3.2 and A3.3. The typically provided by the manufacturer is Other Steel in the AISI Specification and requires the use of material coupons to verify material properties.		
ASTM A653, Grades **33**, 37, 40, **50** **(Most common)**	F_y =228 to 345 Mpa (33 to 50 ksi)	F_u = 310 to 483 MPa (45 to 70 ksi)
ASTM A653 Grade 80 (Deck and Panels)	F_y = 552 MPa (80 ksi)	F_u = 565 MPa (82 ksi)
ASTM A500 Tubes only	F_y =228 to 345 MPa (33 to 50 ksi)	F_u = 310 to 427 MPa (45 to 62 ksi)

d. Effective Width. The major cold-formed steel design concept is the “Effective Design Width” of flat elements. These flat elements can be stiffened, partially stiffened, or unstiffened. Using a bent edge that can form a lip as on a C or Z section stiffens flat elements. Also, using an intermediate crease in the middle of the flat element on the top of a hat section, or using another flat element such as the sides of a hat section. When the stiffening element is not fully effective such as an overly long lip on a C or Z section the flat element is partially stiffened. An unstiffened element would be a C or Z section that does not have an edge lip. Cold-formed steel sections have very high width to thickness ratios which means elements within the section are susceptible to local buckling. Therefore, only that portion of the section that is capable of taking load is included in the effective width of the flat element. Currently, only the AISI approach to determine effective section properties is allowed, other methods that have been used include finite strip analysis and finite element methods.

e. Advantages of Cold-Formed Steel. The advantages of cold-formed steel systems occur because of several unique material characteristics such as: the recyclable nature of the material, low weight, high strength, custom shapes, rounded corners with no fillets, post buckling strength, and element stiffening characteristics.

f. Limit States. Designers should recognize the two limit states used in the AISI specification: the Ultimate Limit State (ULS) and the Serviceability Limit State (SLS). The first limit state is a function of strength, the second is structural function. AISI has very limited coverage of serviceability limit states since they are different for each owner, user, and designer, see Paragraph 4.j. for further guidance on live load deflection limits. Typical design considerations for a member in bending would show how these two limit states are used. The strength limit states for beams would include: flexure, shear, web crippling, and shear lag, while the serviceability limit state would include deflections and flange curling. Many times section will look distressed, buckled or distorted and still have a great deal of strength remaining. Therefore, serviceability limit states are set to reduce distortion effects.

g. Design Thickness. Designers need to calculate the sectional properties using the design or nominal thickness of the material. The AISI design equations do account for the minimum or delivered thickness difference. When using the AISI specification the equation for calculating the nominal strength of the section are the same for Allowable Strength Design (ASD) or Load Factor Resistant Design (LRFD) methods. The method of calculation of section properties is unique to the AISI specification as effective sections properties are used in many of their procedures. Thin materials and high strength steels combine to make sections that are subject to not only local buckling, lateral buckling, but also lateral-torsional buckling, and therefore, sections are not always fully effective. Non-fully effective sections need to have their effective section properties calculated. This calculation is a iterative process for C and Z sections, when the neutral axis is located nearer to the tension flange of the section, and the section properties are calculated based on the compression flange yielding first. The effective section is based on the effective flat widths of the compression-flange, the flange-lip, and the section's web. All sections are to be designed to develop their full strength using an all steel design. This means that all cold-formed steel sections are to be braced with steel to prevent lateral or lateral-torsional buckling created by lateral or twisting loads. Sheathing and gypsum wall board are not allowed to brace the stud section. When the section is fully effective the nominal moment capacity is equal to the fully effective section modulus times the yield strength of the steel. Typically, cold-formed sections are too thin to develop plastic sections and cannot redistribute plastic moments. Also, web crippling is usually required at concentrated loads and at beam supports, and web reinforcement may be required around pipe openings. Around these areas of the beam designers should check the requirements for beam web stiffening with reinforcement plates at pipe opening or web crippling at supports and concentrated loads as necessary.

4. DESIGN OF STRUCTURAL ELEMENTS

a. AISI Specification. The AISI Specification for the Design of Cold-Formed Structural Members-1996 applies to the design of all cold-formed steel members.

b. Preliminary Member Selection. A good report to use when designing cold-formed steel sections is the AISI Report CF 93-1, "Preliminary Design Guide for Cold-Formed C and Z Members", June 1993. This report uses Gross section properties and reduced stresses to size cold-formed sections. It has in it design procedures for strength in bending, strength in shear, strength in combined bending and shear, strength in web crippling, strength of concentrically loaded compression members, and strength in combined axial and bending. Designers using this procedure will normally be conservative by 5 to 15%.

c. Element Behavior. Designers will quickly learn about the characteristics of thin compression elements, elastic buckling, and post buckling strength. Within the AISI criteria stresses are referred to as flat plate elements and plate stresses, these are in reference to the flat rolled elements of the cross section being analyzed. Designers should note that AISI assumes that flat plate stresses are assumed to be uniform across the plate element's width as the plate reaches the critical plate buckling stress, which has a nonuniform distribution. Sine wave ripples along the length of the member characterize buckling of a thin compression element. Thin plate elements go into an elastic buckling mode when the longitudinal plate stresses exceed $f_{cr,}$ the critical buckling stress of the element as calculated by Equation 2-1. As flat elements buckle the plate stresses are redistributed to the stiffened edges of the plate, and edge stresses approach the yield stress of the steel. Therefore, the effective width is a function of the design stress. This redistribution effect was developed by Mr. George Winter at Cornell University, and has been used by the airline industry since the 1940's. This is the basis of the effective width concept used to design cold-formed steel section. The constant λ (lambda) as shown in Equation 2-2 is used to calculate the effective width of the flat element. Values of λ less than or equal to 0.673 indicate a fully effective element, otherwise they are less than fully effective. While such an effect may appear to be detrimental there can still be a significant amount of strength remaining. The elastic critical buckling factor, k of a stiffened element can increased by a factor of 9.3 over an unstiffened element.

$$f_{cr} = \frac{k\pi^2 E}{12(1-\mu^2)\left(\frac{w}{t}\right)^2} \qquad \text{(Eq 2-1)}$$

$$\lambda = \left(\frac{1.052}{\sqrt{k}}\right)\left(\frac{w}{t}\right)\sqrt{\frac{f}{E}} \qquad \text{(Eq 2-2)}$$

when

$$\lambda \leq 0.673... \Rightarrow b = w \qquad \text{(Eq 2-3)}$$

when

$$\lambda > 0.673... \Rightarrow b = \rho w \qquad \text{(Eq 2-4)}$$

$$\rho = \frac{\left(1 - \frac{0.22}{\lambda}\right)}{\lambda} \qquad \text{(Eq 2-5)}$$

Where:

- λ = an element slenderness factor
- k = a plate buckling coefficient = 4 for a stiffened element, and 0.43 for an unstiffened element
- w = the flat width of the element
- t = the thickness of the element
- f = the design stress in the element determined at the Nominal Moment (M_n) based on the effective section properties
- f_{cr} = the critical elastic buckling stress for the plate
- E = the modulus of elasticity of the element
- ρ = an effective width factor
- μ = Poisson's ratio (0.3 for steel) in the elastic range

d. Element Slenderness. A review of the slenderness factor shows the relationship of the width to thickness ratio to design stress. Very thin members are less effective at higher stresses. The AISI specification limits element w/t ratios to be less then 60 for flanges that are stiffened with simple lips (C and Z sections), and flanges that are stiffened with elements that are stiffened (hat sections) to less than 500. When elements have a w/t ratio greater than 30 and 250 they will display a noticeable waviness before reaching their design capacity, this does not effect the members final design capacity.

e. Simplified Section Properties. A simplified approach to calculating section properties uses the centerline of each section element. Section properties are calculated by using the corners and flat element dimensions. When calculating the moment of inertia the corners are so small they can be ignored. When a lip is used to stiffened an element the moment of inertia of the edge stiffener must be greater than the required moment of inertia per AISI Section B2, *Effective Widths of Stiffened Elements*. When the neutral axis is closer to the tension flange or at mid-depth on a symmetrical section, the design stress in the tension flange is at yield, and λ is calculated based on the compression stress in the flange. Should the neutral axis be closer to the compression flange than the solution becomes iterative to locate the neutral axis, compressive stress, and λ.

f. Members.

(1) Properties of Sections, Section C1. Typically full cross section properties are used except where reduced or effective element widths are required. A computer program is needed to efficiently calculate effective section properties

(2) Tension Members, Section C2. Common rules for hole placement and largest hole reduction is in AISI specification. Currently there are no shear lag provisions in the specification. However, there will be in the future manual. Also, A307 Bolts are commonly used in Cold-Formed Steel design.

(3) Flexural Members, Section C3.

(a) Strength for Bending Only, Section C3.1. Reference Yura on member bracing. For screw down roof systems, brace the C's and Z's at 1/3 points for uplift loads only.

(b) Strength for Shear Only, Section C3.2

(c) Strength for Combined Bending and Shear, Section C3.3.

(d) Web Crippling Strength, Section C3.4.

(e) Combined Bending and Web Crippling Strength, Section C3.5.

(4) Concentrically Loaded Compression Members, Section C4. Torsional-Flexural Buckling is unique to cold-formed columns and beam-columns. and the Factor of Safety for Columns is equal to 1.80. Torsional-Flexural Buckling controls most columns.

(a) Sections Not Subject to Torsional or Torsional-Flexural Buckling, Section C4.1.

(b) Doubly or Singly Symmetric Sections Subject to Torsional or Torsional – Flexural Buckling, Section C4.2

(c) Nonsymmetrical Sections, Section C4.3.

(d) Compression Members Having One Flange Through-Fastened to Deck or Sheathing, Section C4.4.

(5) Combined Axial Load and Bending, Section C5.

(a) Combined Tensile and Axial Load and Bending, Section C5.1.

(b) Combined Compressive Axial Load and Bending, Section C5.2

(6) Cylindrical Tubular Members, Section C6.

(a) Bending, Section C6.1.

(b) Compression, Section C6.2

(c) Combined Bending and Compression, Section C6.3.

g. Wall Studs and Wall Stud Assemblies. Section D4. Use only all steel designs. Therefore only paragraph (a) is allowable. The effective area, A_e and the nominal buckling stress, F_n are to be calculated in accordance with Section B.

(1) Wall Studs in Compression. Section D4.1. Studs need to be checked for column buckling between fasteners (a) and flexural and or torsional column buckling (b). Paragraph (c) is not to be used since it deals with sheathing for bracing. Studs used in wall sections will be firmly placed in the track prior to attachment of the stud track units. No gaps will be allowed between the stud web/flange and the track being assembled.

(2) Wall Studs in Bending. Section D4.2. Wall studs should always have stiffened or partially stiffened compression flanges. Ignore the values shown for unstiffened compression flanges. The provisions of Section C3.1 apply to the bending strength of the member except for Section C3.1.2 Lateral Buckling Strength. Calculations should be based on stiffened and partially stiffened compression flanges when determining the nominal moment capacities M_{nxo} and M_{nyo}. Anchor bridging to solid blocking is critical

(3) Wall Studs with Combined Axial Load and Bending. Section D4.3. Interaction equations for this loading condition are in Section C5.

(4) Floor, Roof or Wall Steel Diaphragm Construction. Section D5. This section provides design values for various diaphragm conditions.

h. Design Guide for Cold-Formed Steel Trusses - AISI RG-9518. Trusses will be designed using the following guidance.

(1) Member ends will be assume to be pinned.

(2) Webs of members are to be pinned.

(3) Unbraced lengths will be:

(a) Continuous chord members,

- K_x, K_y, K_t = 0.75 when appropriate sheathing is attached to the top and bottom flange of those chords.
- K_x, K_y, K_t for other conditions = unity or 1.0.

(b) Compression web members K = 1.0.

(c) End moment coefficient, C_m = 0.85, and strength amplification factor, C_b per AISI C3.1.2

(d) Compression chord with combined axial and bending use section C5 at panel points Equation C5.2.1-2, and between panel points equation C5.2.1-7.

Where:

L_x, L_t = panel length
L_y = distance between sheathing connections

i. Shear Wall Design Guide - AISI RG-9604 and Chapter 3 of this manual.

(1) Braced shear wall systems will be designed for strength and stiffness as follows. The greater requirement will control the size of the bracing system. Also, the designer will consider the seismic bracing guidance in Chapter 3 of this technical instruction.

(2) Strength requirements for a brace force of, F_{br} as follows for ASD or LRFD design methods:

$$F_{br} = 0.004P\left(L/B\right) \qquad \text{(Eq 2-6)}$$

Where:

F_{br} = Brace force in consistent units.
P = Share of the gravity load supported by the shear wall frame. Should two frames support the entire floor load P is ½ the floor load.
L = Length of diagonal brace.
B = Width of the frame bent.

(3) Brace Stiffness will meet the following requirement:

$$A_b = \frac{C(P)(L^3)}{E(h)(B^2)} \qquad \text{(Eq 2-7)}$$

Where:

A_b = Cross sectional area of the brace in consistent units.
C = Constant: ASD = 4, LRFD = 2.67.
P = Share of gravity load supported by the braced frame for lateral stability.
L = Length of diagonal brace.
E = 29,500 Ksi, or 203 395 MPa.
h = Height of braced frame.
B = Width of braced frame.

j. Serviceability Deflection Limits

(1) Live Load Deflections. Deflection criteria for buildings under load are generally established to ensure functional performance and economy of design. Many of the deflection limits used today have been established based on past performances and the perception of the occupants in a building. A consideration for deflection limits is often tied into the performance of the attached claddings. A stiffer cladding requires a stiffer backup to prevent damage to the cladding. In the case of floor joists, the limit may be established to reduce vibrations (see paragraph 2.4.m) which are unacceptable to the occupants. In any case, consideration should be given to all serviceability issues. The following list is a compilation of suggested deflection limits by manufacturers, trade groups and building officials that may be applied to light gauge metal framed structures. The list below assumes limits at full dead plus live load; full dead plus snow; or full dead plus wind load unless noted otherwise.

Exterior wall finishes

Brick veneer over sheathed framing	L/600
Stucco over sheathed or open framing	L/360
Exterior Insulation Finish Systems	L/240
Cement board sheathing over framing	L/360
Stone (marble, granite, limestone)	VERIFY WITH STONE SUPPLIER
Plywood and Oriented strand board	L/240
Gypsum sheathing over framing	L/240
Metal or vinyl siding over sheathed framing	L/240

Interior wall finishes

Plaster	L/360
Ceramic tiles	L/360

Gypsum drywall	L/240
Wood finishes	L/240

Floors

Residential	L/360 total load; L/480 live loads
Office and storage	L/240 total load; L/360 live loads

Roofs

Trusses	L/240 total load; L/360 live load
Rafters	L/180 total load; L/240 live load

(2) Drift Limits. See Chapter 3 for drift limits guidance when using cold-formed steel shear walls. Deflection states for buildings broadly encompass a variety of design considerations the designer should be aware of but are not covered in this brief synopsis. The deflection limits stated above are the limit states for various building elements under load. There are also limits for lateral deflections (story drifts) due to wind or seismic loading of the building, these are best determined by review of the applicable model building code. Another consideration for the designer is long term deflection (creep). While this often does not effect the design, creep should be considered when a large proportion of the total load is do to permanent (dead) loads.

k. Continuous Beams and Joists. *"Effective Lengths for Laterally Unbraced Compression Flanges of Continuous Beams Near Intermediate Supports"* by J. H. Garrett, Jr., G. Haaijer, and K. H. Klippstein, Proceedings, Sixth Specialty Conference on Cold-Formed Steel Structures, University of Missouri-Rolla / American Iron and Steel Institute

l. Effect of Holes. Proposed additions to the Specification for the Design of Cold-Formed Steel Structural Members and accompanying Commentary, Sections B2.4, C3.2.2, and C3.4.2, based on University of Missouri-Rolla, Department of Civil Engineering, Reports on Behavior of Cold-Formed Steel Sections with Web Openings, by Roger A. LaBoube. Table 2-3 shows the typical dimensions of holes that can be placed in various size joist members.

Table 2-3: Pipe Openings: Maximum Pipe Opening and Web Reinforcement

Joist Size with 41.3 mm (1-5/8") Flange mm (in)	"d" Max hole Diameter mm (in)	"b" Plate Size mm (in)	"a" Screw Hole Spacing mm (in)	"c" Edge Distance mm (in)	"A" Min Distance to Concentrated Load or Support
292 (11.5)	175 (6-7/8)	229 (9)	67 (2-5/8)	14 (9/16)	L/6
235 (9.25)	133(5-1/4)	229 (9)	57 (2-1/4)	24 (15/16)	L/25
203 (8.0)	108 (4-1/4)	178 (7)	48 (1-7/8)	17 (11/16)	L/16
184 (7.25)	108 (4-1/4)	178 (7)	48 (1-7/8)	17 (11/16)	L/16
152 (6.0)	89 (3-1/2)	133 (5-1/4)	37 (1-7/16)	12 (15/32)	L/10

Notes:
1. L = joist span.
2. b = reinforcement plate dimensions for a square plate.
3. a = spacing between screw holes that attach the reinforcement plate to the joist.
4. c = Minimum edge distance of screws from the edge of the reinforcement plate.

m. Floor Vibrations. Light weight floors and short spans can lead to vibration problems in floor systems. The following methods uses the dimensions and cross-sectional properties of the floor and the predicted central floor deflection. This procedure was developed by Kraus and Murray for residential floor systems.

(1) Calculate the critical central floor deflection, y_{crit} (in) from Onysko's Criterion (Onsyko 1995):

$$y_{crit} = \frac{37.32}{L^{1.3}} \qquad \text{(Eq 2-8)}$$

Where:

L = Floor span in inches

(2) Calculate the predicted deflection of a single joist, Δ_{ot} due to a 255 lb concentrated load at midspan:

$$\Delta_{ot} = \frac{255L^3}{48EI} \qquad \text{(Eq 2-9)}$$

Where:

L = Floor span in inches.
E = Modulus of Elasticity of the joist.
I = Moment of Inertia of a single joist.

(3) Calculate the number of effective joist, N_{eff} from the Steel Joist Institute (SJI) equation:

$$N_{eff} = 1 + 2\sum\left(\cos\frac{x\pi}{2x_o}\right) \qquad \text{(Eq 2-10)}$$

Where:

x = Distance from the center joist to the joist under consideration (inches).
x_o = Distance from center joist to the edge of the effective floor = 1.06eL (inches).
L = Joist span (inches).
$e = (D_x/D_y)^{0.25}$
D_x = Flexural stiffness perpendicular to the joist = $E_c t^3/12$
D_y = Flexural stiffness parallel to the floor joists = EI_t/S
E_c = Modulus of elasticity of the sub-flooring.
E = Modulus of elasticity of the joists.
t = sub-flooring thickness.
I_t = Moment of inertia of joists alone.
S = Joist Spacing.

(4) Calculate the predicted central floor deflection, Δ_o:

$$\Delta_o = \frac{\Delta_{ot}}{N_{eff}} \qquad \text{(Eq 2-11)}$$

Where:

Δ_o = Deflection of floor at mid-bay.
Δ_{ot} = Deflection of a single joist due to 255 lb. concentrated load at midspan.
N_{eff} = Number of effective joists in the floor system.

(5) Compare the Δ_o value to the critical deflection, y_{crit}:

If	$\Delta_o < y_{crit}$:	Acceptable
If	$y_{crit} < \Delta_o \le 1.1\ y_{crit}$:	Marginal
If	$\Delta_o > 1.1\ y_{crit}$:	Unacceptable

5. FASTENERS AND CONNECTIONS

a. Sheet Metal Screws. AISI Specification for the Design of Cold-Formed Steel Structural Members and accompanying Commentary - Section E4. This section is applicable to screws with a nominal diameter of 2.03 mm (0.08 in) $\leq$ d $\leq$ 6.35 mm (0.25 in). The nominal diameter is measured across the threads and will be thread forming or thread cutting. Screws may be used with or without a self drilling point. Table 2-4 gives some suggested loading values for screw connections. Pullout values are for attaching facing materials and are not to be used for connection design.

b Bolts. AISI Specification for the Design of Cold-Formed Steel Structural Members and accompanying Commentary - Section E3. Bolts are designed for sheets with the thinnest sheet being less than 4.8 mm (3/16 in). When the thinnest sheet is greater than 4.8 mm (3/16 in) use the AISC specification. Four design conditions need to be considered:

- Longitudinal shearing of the sheet parallel to the through the end of the sheet,
- Bearing or piling up behind the bolt,
- Tearing through the net section,
- Shearing of the bolt.

Table 2-4: Suggested Capacities for Screw Connectors in kN (lbs)

Steel Nominal / Design Thickness mm (in)	No. ¼-14		No. 12-14		No. 10-14		No. 8-14		No. 6	
	Shear or Bearing	Pullout	Shear or Bearing	Pullout	Shear or Bearing	Pullout	Shear or Bearing	Pullout	Shear or Bearing	Pullout
2.583 (0.1017)	2.60 (585)	1.57 (352)	2.00 (450)	1.44 (324)	1.45 (327)	1.40 (314)	NA	1.35 (303)	NA	NA
1.811 (0.0713)	2.27 (511)	1.08 (242)	1.83 (412)	0.96 (215)	1.27 (286)	0.91 (205)	NA	0.89 (200)	NA	NA
1.438 (0.0566)	1.89 (426)	0.71 (159)	1.68 (377)	0.68 (153)	1.16 (261)	0.67 (151)	1.05 (236)	0.63 (142)	NA	0.59 (132)
1.146 (0.0451)	1.34 (301)	0.45 (101)	1.23 (276)	0.45 (101)	1.17 (263)	0.44 (98)	1.10 (248)	0.42 (94)	0.84 (188)	0.37 (83)
0.879 (0.0346)	0.69 (154)	0.32 (71)	0.64 (143)	0.31 (70)	0.63 (141)	0.31 (69)	0.62 (140)	0.30 (68)	0.59 (133)	0.24 (53)

Notes:
1. NA: not applicable, two thicknesses of this metal gage cannot be connected by this size screw.
2. Screw capacity is based on a minimum connected strength of F_y = 228 MPa (33 ksi). The ratio of the material ultimate tensile strength to yield strength should be equal to or greater than 1.15.
3. Screw a spacing and edge distance shall not be less than 1.5D or P/0.6F_{yt}, where D is the screw shank diameter and P is the shear load.
4. Screw capacities are based on average test results divided by a safety factor of 3.0. Test data is available from Buildex Division of ITW, Inc Itasca, Illinios; test #845, uninspected values.
5. For steels having yields other than 228 MPa (33 ksi), use the following formula:
Table Value(Actual yield strength)/(228 or 33 in consistent units) = New value

c. Welds. AISI Specification for the Design of Cold-Formed Steel Structural Members and accompanying Commentary - Section E2. The maximum thickness for the use of the AISI specification when welding sheets together is 4.6 mm (0.18 in) for the thinnest sheet. When welding thicker sheets use the AISC specification. Resistant welded sheets are limited to 3.2 mm (0.125 in) or less for the thinnest sheet. Table 2-5 gives some suggested values for fillet and flare-groove welds.

Table 2-5: Suggested Design Loads for Fillet and Flare-Bevel Groove Welds

Design Thickness t mm (in)	Weld Size mm (in)	Weld Strength	
		Fillet N/mm (lbs/in)	Flare-Bevel Groove N/mm (lbs/in)
3.15 mm (0.1240")	4.76 (3/16)	215 (1228)	172 (982)
2.583 (0.1017)	3.97 (5/32)	176 (1007)	141 (806)
1.811 (0.0713)	3.18 (1/8)	124 (706)	99 (565)
1.438 (0.0566)	3.18 (1/8)	98 (560)	78 (448)
1.146 (0.0451)	3.18 (1/8)	78 (447)	63 (358)

Notes:
1. Welds can be positioned in shear or tension.
2. Weld strength for fillet = 0.3 F_y t, where t = minimum welded material thickness.
3. Weld Strength for flare-bevel groove = 0.3 F_y t/1.25.
4. Values shown are for F_y = 228 MPa (33 ksi). For F_y = 278 MPa (40 ksi) multiply tabulated values by 1.33. For F_y = 345 MPa(50 ksi) multiply tabulated values by 1.52.
5. Flare-bevel groove welds occur between the outside radius of one piece and a flat surface of another piece.

d. Anchors. ASTM F 1554 for Anchor Bolts covers straight, bent, headed, and headless bolts for anchoring the structural support to the foundation. Bolts covered have diameters from 6.35 mm (¼ in) to 101.6 mm (4 in) and yield strengths of 248, 379, and 724 MPa (36, 55 and 105 ksi).

i) Expansion Anchors or similar devices will be designed as bolted connection between the anchor and the cold-formed structure. In lieu of specific anchor data the suggested values in table 2-6 may be used. Designer must assure that the anchors as supplied meet the design requirements.

Table 2-6: Suggested Capacity for Expansion Anchors in Stone Aggregate Concrete

Anchor Diameter mm (in)	Minimum Embedment mm (in)	Type of Loading	Concrete Strength MPa (psi)			Minimum Anchor Spacing mm (in)	Minimum Edge Distance mm (in)
			13.8 (2000) kN (lbs)	27.6 (4000) kN (lbs)	41.4 (6000) kN (lbs)		
6.35 (¼)	64 (2-1/2)	Pullout Shear	1.45 (325) 1.69 (380)	1.89 (420) 2.89 (650)	1.87 (420) 2.89 (650)	64 (2-1/2)	32 (1-1/4)
13 (½)	70 (2-3/4)	Pullout Shear	2.96 (665) 7.6 (1710)	4.00 (900) 9.25 (2080)	5.38 (1210) 10.3 (2320)	127 (5)	64 (2-1/2)
19 (¾)	83 (3-1/4)	Pullout Shear	4.16 (935) 13.6 (3050)	5.65 (1270) 19.0 (4270)	6.0 (1360) 20.0 (4510)	191 (7-1/2)	95 (3-3/4)
25 (1)	114 (4-1/2)	Pullout Shear	7.2 (1610) 27.9 (6280)	8.9 (2000) 29.9 (6720)	11.3 (2530) 35.4 (7960)	254 (10)	127 (5)

Notes:
1. Pullout values listed may be doubled with special inspection.
2. Values may not be increased 1/3 for wind for seismic loads.
3. ICBO uninspected values - Hilti/ICBO report #2156.

ii) Powder driven pins may be used to attach cold-formed members to concrete or structural steel typical suggested capacities are shown in tables 2-7 and 2-8.

e. Connections and Joints. In Cold-formed steel design the AISI specification is to be used when the thickness of the thinnest sheet being connected is less than 4.8 mm (3/16"), when the thickness of the thinnest material is greater than 4.8 mm (3/16") use the AISC specification. AISI uses only bearing connections, snug tight fit. In cold-formed design washers are typically not used in cold-formed steel construction. The primary mode of failure is sheet tearing for AISI designed connections. Two limit states are considered in the design of cold-formed steel connections: tilting of screw and subsequential tearing of sheet, and tension pull over are common. Riveted and crimped fasteners are proprietary connections, they are not to be used in COE designs. Table 2-9 gives some suggested loads per fastener for joist clip angles.

Table 2-7: Suggested Capacity for Powder Driven Fasteners in Concrete

Shank Diameter Mm (in)	Minimum Penetration mm (in)	Type of Loading	Concrete Compressive Strength MPa (psi) 13.8 (2000) N (lbs)	20.7 (3000) N (lbs)	27.6 (4000) N (lbs)
3.68 (0.145)	29 (1-1/8)	Pullout Shear	0.40 (90) 0.71 (160)	0.51 (115) 1.00 (225)	0.65 (145) 1.18 (265)
4.50 (0.177)	37 (1-7/16)	Pullout Shear	0.67 (150) 1.11 (250)	0.91 (205) 1.27 (285)	1.22 (275) 1.47 (330)
5.21 (0.205)	32 (1-1/4)	Pullout Shear	0.98 (220) 1.74 (390)	1.25 (280) 1.98 (445)	1.54 (345) 2.22 (500)

Notes:
1. Capacities shown are for stone aggregate concrete and are based on a low velocity shot.
2. Minimum fastener spacing: 4"; minimum fastener edge distance: 3".
3. Values may not be increased by 1/3 for wind or seismic loads.
4. ICBO uninspected values - Hilti/ICBO research #2388.

Table 2-8: Suggested Capacity for Powder Driven Fasteners in Structural Steel

Steel Thickness	Shank Dia: 3.68 mm (0.145")			Shank Dia: 4.50 mm (0.177")			Shank Dia: 5.21 mm (0.205")		
mm (in)	6.35 mm (¼") kN (lbs)	9.53 mm (3/8") kN(lbs)	13 mm (½") kN(lbs)	6.35 mm (¼") kN(lbs)	9.53 mm (3/8") kN(lbs)	13 mm (½") kN(lbs)	6.35 mm (¼") kN(lbs)	9.53 mm (3/8") kN(lbs)	!3 mm (½") kN(lbs)
2.583 (0.1017)	0.93 (210)	0.93 (210)	0.93 (210)	1.49 (335)	1.76 (395)	1.76 (395)	2.16 (485)	2.34 (525)	2.94 (660)
1.811 (0.0713)	0.93 (210)	0.93 (210)	0.93 (210)	1.49 (335)	1.76 (395)	1.76 (395)	2.16 (485)	2.34 (525)	2.58 (581)
1.438 (0.0566)	0.93 (210)	0.93 (210)	0.93 (210)	1.49 (335)	1.76 (395)	1.76 (395)	2.16 (485)	2.07 (465)	2.07 (465)
1.146 (0.0451)	0.93 (210)	0.93 (210)	0.93 (210)	1.43 (321)	1.43 (321)	1.43 (321)	1.65 (372)	1.65 (372)	1.65 (372)
0.879 (0.0346)	0.88 (197)	0.88 (197)	0.88 (197)	1.10 (247)	1.07 (241)	1.07 (241)	1.24 (279)	1.24 (279)	1.24 (279)

Notes:
1. Tests were conducted with the fastener point driven completely through the back side of the hot-rolled steel member. This was necessary to obtain proper gripping force.
2. Fasteners should not be located less than 13 mm (1/2") from the edge of steel.
3. A minimum fastener spacing of 38 mm (1-1/2") is necessary.
4. Bearing strength based upon: 1.15(228 MPa)(Bearing Area) [1.15(33ksi)(Bearing Area)] for cold-formed steel.
5. Capacities shown are for either shear or pullout.
6. Values may not be increased by 1/3 for wind or seismic loads.
7. ICBO uninspected values - Hilti/ICBO research #2388

Table 2-9: Joist End Clip: Allowable Loads per fastener

Clip Length mm (in)	No. of screws in Each Leg (#10-16)	Joist Thickness 0.8879 mm (0.0346") 228 MPa (33 ksi) kN (lbs)	1.146 mm (0.0451") 228 MPa (33 ksi) kN (lbs)	1.438 mm (0.0586") 228 MPa (33 ksi) kN (lbs)
152 (6)	3	1.50 (337)	2.23(501)	4.76 (1070)
203 (8)	4	2.14 (480)	3.17 (713)	6.77 (1523)
254 (10)	5	2.78 (626)	4.14 (930)	8.83 (1985)

Notes:
1. Based on CCFSS technical bulletin vol. 2, no. 1 which outlines the proposed AISI specification provisions for screw connections.
2. F_y = 228 MPa (33 ksi) for 0.879 mm (0.0346") sheets. F_y = 345 MPa (50 ksi) for 1.438 mm (0.0566") sheets. $F_u = 1.08F_y$
3. Allowable loads based on a factor of safety of 3.0.

f. Fasteners. Information on Fasteners for Residential Steel Framing can be found in AISI RG-933.

6. USEFUL RELEVANT INFORMATION

a. Beam Diagrams and Formulas

AISC Steel Construction Manual

b. Material Weights

AISC Steel Construction Manual

c. Software. Listings of available cold-formed steel design software can be found through the CCFSS http://www.umr.edu/~ccfss/ Technical Bulletin, Center for Cold-Formed Steel Structures, University of Missouri Rolla.

CHAPTER 3

SEISMIC DESIGN GUIDANCE FOR SHEAR WALLS (DIAGONAL STRAP SYSTEMS)

1. INTRODUCTION. The design guidance presented here is tied directly to the 1997 NEHRP (FEMA 302 and 303), because this will form the basis of the 2000 International Building Code. U.S. Army Corps of Engineers, Seismic Design for Buildings, TI 809-04 is the general military standard for seismic design of buildings, and this is also based on FEMA 302 and 303. TI 809-04 supplements the FEMA 302 and 303 with additional guidance for military buildings that is primarily based on the 1997 NEHRP Guidelines for the Seismic Rehabilitation of Buildings (FEMA 273 and 274). These and other TI 809-04 guidance that differ from the FEMA 302 and 303 are summarized in paragraph 2 of this chapter. The basis for unique seismic guidance presented here is provided in technical report found at the URL address: http://owww.cecer.army.mil/techreports/wilcfstr.post.pdf , Development of Cold-Formed Steel Seismic Design Guidance, U.S. Army Construction Engineering Research Laboratory. Design guidance is also based on the following references:

- Cold Formed Steel Design Manual, American Iron and Steel Institute, 1996 Edition.
- Manual of Steel Construction Load and Resistance Factor Design (LRFD), American Institute of Steel Construction (AISC), 2nd Edition, 1994.
- Seismic Provisions for Structural Steel Buildings, AISC, 1997.
- Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers (ASCE) 7-95, 1995.
- State-of-the-Art Report on Anchorage to Concrete, American Concrete Institute (ACI) 355.1R-91, 1991.

Unique guidance for cold-formed steel is included in this chapter, while guidance that remains unchanged from FEMA 302 and other standards is included in Appendix C as indicated in the paragraphs that follow. Appendix C also contains limited background on the development of the guidance provided in this chapter. The source of all guidance is referenced in the text. Figure 3-1 gives a flowchart for seismic design of cold-formed steel shear walls. Appendix D presents an example problem showing the application of Chapter 3 and Appendix C seismic design guidance. An update to the spreadsheet, http://owww.cecer.army.mil/techreports/wilcfsxl.post.pdf design program using the example problem is available.

2. TI 809-04, SEISMIC DESIGN FOR BUILDINGS. TI 809-04 is the military standard for the seismic design of buildings, and it provides additional guidance primarily based on FEMA 273 and 274. Additional guidance related to the design of cold-formed steel buildings is summarized as follows:

- Classification of Buildings: Definitions of Seismic Use Groups (Table 4-1) and Seismic Design Categories are expanded (Table 4-2a and 4-2b). The Seismic Use Groups are used in TI 809-04 to define Performance Objectives.
- Ground Motion: Ground Motion A is the FEMA 302 defined 2/3 site adjusted maximum considered earthquake (MCE) levels. TI 809-04 defines another ground motion level, Ground Motion B, which is defined as ¾ of the same MCE levels.
- Performance Objectives: TI 809-04 defines three performance levels: 1) Life Safety, 2) Safe Egress, and 3) Immediate Occupancy defined in Table 4-3. These levels are combined with the two design motion levels to define performance objectives for each of the four seismic use groups as described in Table 4-4. These objectives are 1A (Life Safety), 2A, 2B and 3B.
- Minimum Analytical Procedures: Chapter 5 defines three analytical procedures and the minimum procedure that must be used for each performance objective. Linear analysis with response modification factor, R as described in FEMA 302 is used for performance objective 1A. Linear analysis may also be used for Performance Objectives 2A, 2B and 3B, but with a modification factors, m for deformation controlled structural components or elements.

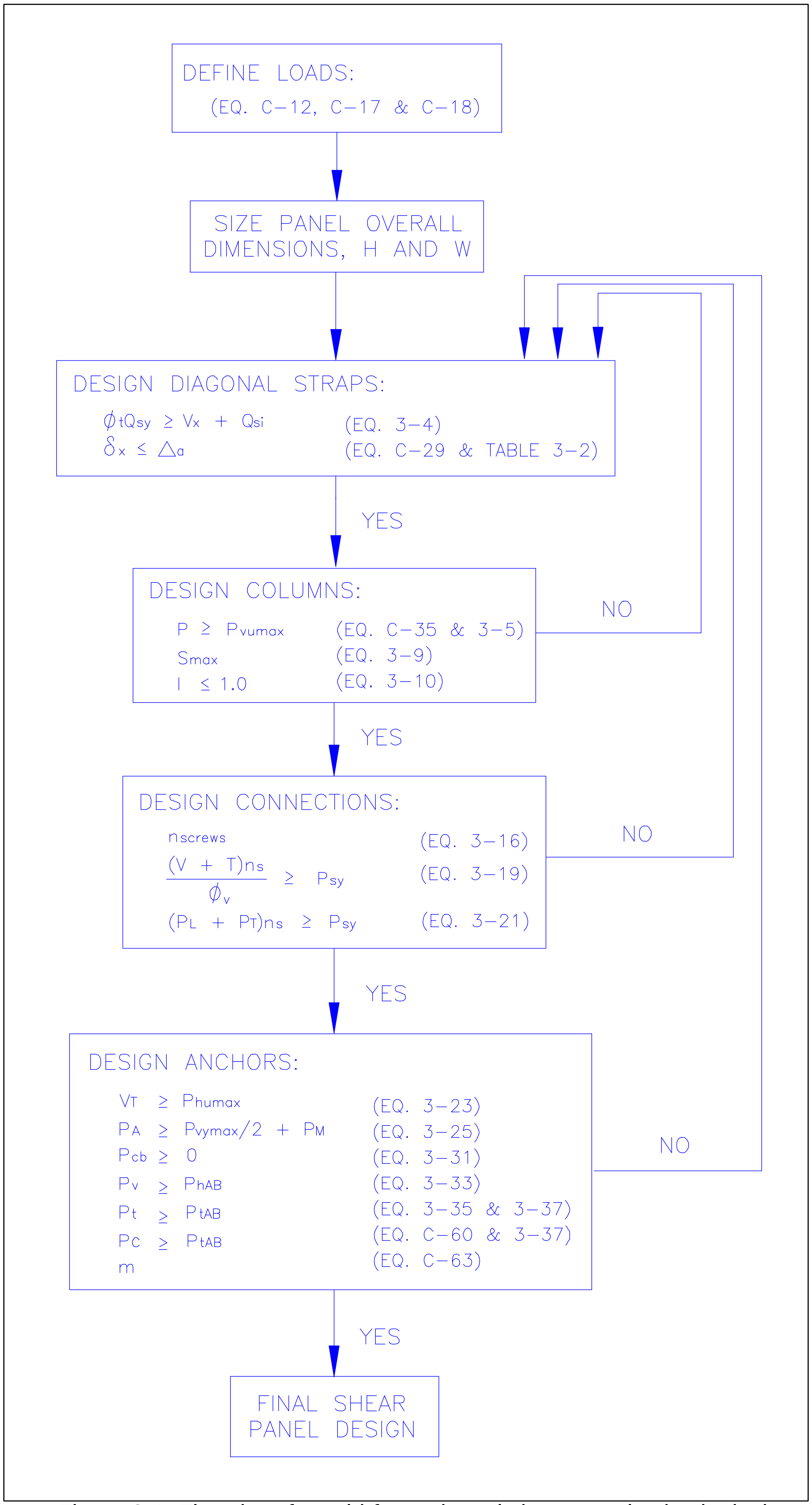


Figure 3-1. Flowchart for cold-formed steel shear panel seismic design.

- Acceptance Criteria: Acceptance criteria for each performance objective are prescribed for each analytical procedure in Chapter 6 and numerical values for each of the criteria are given in Chapters 7 through 10.

3. STRUCTURAL DESIGN CRITERIA. The basic lateral and vertical seismic-force-resisting systems considered here are diagonal strap configurations (Panels A1, A2, and D1) in Appendix B. These are considered bearing wall systems. The format of Table 5.2.2 of FEMA 302 is used in Table 3-1 to present the response modification coefficient, R and deflection amplification factor, C_d. These values are used to calculate the base shear, and design story drift. The system overstrength factor, Ω_0 used in FEMA 302 is not included here because shear panel overstrength is accounted for by $\Omega_0 Q_E$ in Equation C-16. This is the maximum lateral capacity of the shear panel based on the maximum estimated ultimate stress of the panel diagonal straps.

The response modification coefficient, R in the direction under consideration at any story shall not exceed the lowest value for the seismic-force-resisting system in the same direction considered above that story excluding penthouses. Other structural systems (dual systems) may be used in combination with these cold-formed steel panels, but then the smallest R value for all systems in the direction under consideration must be used for determining the loads applied to the entire structure in that direction. Dual systems must be used with caution, particularly if differences in stiffness result in interaction effects (FEMA 302, 5.2.2.4.2) or deformation compatibility problems (FEMA 302, 5.2.2.4.2). Another structural system may be used in the orthogonal direction with different R values, and the lowest R value for that direction only shall be used in determining loads in that orthogonal direction.

Table 3-1 Design Coefficients and Factors for Basic Seismic-Force-Resisting Systems

Basic Seismic-Force-Resisting System	Response Modification Coefficient, R	Deflection Amplification Factor, C_d	System Limitations and Building Height Limitations (ft) by Seismic Design Category				
			B	C	D	E	F
Bearing Wall Systems							
Cold-Formed Steel Shear Panels with Diagonal Strapping	4	3½	NL	NL	65	65	65

4. DEFLECTION AND DRIFT LIMITS. The design story drift, Δ shall not exceed the allowable story drift, Δ_a as obtained from Table 3-2 (FEMA 302, Table 5.2.8), for any story. The design story drift shall be computed as the difference of deflections at the center of mass at the top and bottom of the story under consideration, as determined by Equation C-29 (FEMA 302, Eq 5.3.7.1). For structures with significant torsional deflections, the maximum drift shall include torsional effects. All portions of the structure shall be designed and constructed to act as an integral unit in resisting seismic forces unless separated structurally by a distance sufficient to avoid damaging contact under total deflection, δ_x, as determined by Equation C-29, (FEMA 302, 5.2.8).

Table 3-2 Allowable Story Drift, Δ_a (mm or inches)

Structure	Seismic Use Group		
	I	II	III
Structures with diagonal strap shear walls	$0.020h_{sx}$[1]	$0.015h_{sx}$	$0.010h_{sx}$

[1] h_{sx} is the story height below level x.

5. TORSION. The distribution of lateral seismic forces shall take into account the effects of torsional moment, M_t resulting from the location of masses relative to the center of rigidity (stiffness) of the lateral force resisting frames in both orthogonal directions (FEMA 302, 5.3.5). This torsional moment shall include the effects of accidental torsional moment, M_{ta} caused by an assumed offset of the mass. This offset shall be equal to 5 percent of the dimension of the structure orthogonal to the direction of the applied seismic force. Similar to the lateral seismic forces, the torsional moments, M_t are distributed along the floors of the building according to the vertical distribution factor given in Equation C-26.

The torsional resistance comes from each of the shear wall panels, and the resistance from each panel is proportional to the square of the distance from the center of resistance to the plane of the panel. For a given panel the additional shear force due to torsion, Q_{si} can be expressed as:

$$Q_{si} = k_{si}\Delta_i = k_{si}\rho_i\theta \qquad \text{(Eq 3-1)}$$

Where:

k_{si} = the shear stiffness of shear panel i, and is defined as follows:

$$k_{si} = En_s b_s t_s\left(\frac{W}{H^2 + W^2}\right) \qquad \text{(Eq 3-2)}$$

Δ_i = the lateral in-plane shear deflection of panel i.
ρ_i = the distance from the center of resistance to panel i, perpendicular to the plane of the panel.
θ = the torsional rotation of the building at the floor level above the panel.
E = the modulus of elasticity of steel, equal to 200,000 MPa (29,000 ksi).
n_s = the number of diagonal straps.
b_s = the width of the diagonal straps.
t_s= the thickness of the diagonal straps.
W = the overall panel width.
H = the overall panel height (see Figure 3-2 for a schematic panel drawing showing W and H).

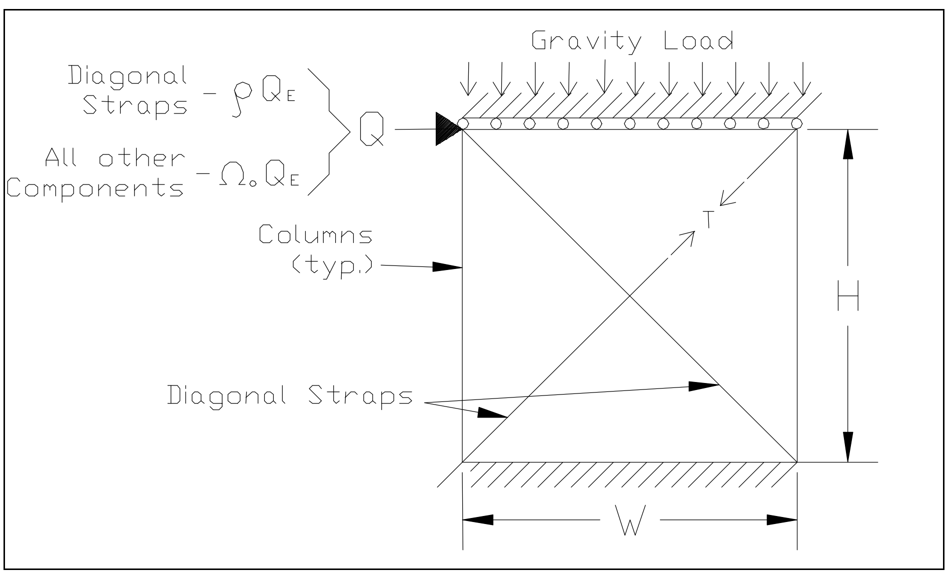


Figure 3-2. Schematic of CERL cold-formed steel shear panel model.

The torsional moment resistance, M_{tr}, for all the shear panels is given by:

$$M_{tr} = \sum_{i=1}^{n} \rho_i Q_{si} = \sum_{i=1}^{n} \rho_i^2 k_{si} \theta \qquad \text{(Eq 3-3)}$$

Equation 3-3 shows that the torsional resistance from each panel is proportional to $\rho_i^2 k_{si}$. The total torsional moment resistance, M_{tr}, is set equal to the M_t and the additional shear force due to torsion, Q_{si} is calculated using Equations 3-1 and 3-3. Note that the torsional rotation, θ in these equations does not need to be solved for and can be treated as a constant. Also the panel shear stiffness, k_{si}, is not needed if all the panels can be assumed to be equal or if their relative stiffness can be determined.

6. COLD FORMED STEEL SEISMIC REQUIREMENTS

a. Wind and Earthquake Loads. The requirements of the 1996 AISI[2], Section A5.1.3, shall be modified as follows (FEMA 302, 8.5.1): "A4.4 Wind or Earthquake Loads where load combinations specified by the applicable code include wind loads, the resulting forces are permitted to be multiplied by 0.75. Seismic load combinations shall be as determined by these provisions."

b. Boundary Members, Chords and Collectors. All boundary members, chords, and collectors shall be designed to transmit the specified induced axial forces (FEMA 302, 8.6.1). Connections for diagonal straps-to-column and columns-to-anchors and shear panel anchorage, and collectors shall have adequate strength to account for the effects of material overstrength as indicated in this guidance. The pullout resistance of screws shall not be used to resist seismic forces (FEMA 302, 8.6.2).

c. Shear Panel Anchors. Shear panels shall be anchored such that the bottom and top tracks are not required to resist uplift forces by bending of the track or track web (FEMA 302, 8.6.3). Both flanges of studs shall be braced to prevent lateral torsional buckling.

d. Pretension of Diagonal Straps. Provision shall be made for pretensioning or other methods of installation of tension-only diagonal straps, to guard against loose straps (FEMA 302, 8.6.4).

e. All Steel Design. The guidance of FEMA 302, 8.6.5 for shear walls shall not be used and configurations with plywood sheathing or oriented strand board are not permitted. The following guidance shall be used in place of the FEMA 302 guidance in section 8.6.5.

Shear panel design shall be based on the cold-formed steel shear design guidance presented here. This design requires that shear panels be adequately anchored at their top and bottoms to a floor diaphragm. Shear panels in the two orthogonal directions must be anchored to the same diaphragm at each floor level to tie the two orthogonal lateral load-resisting systems together. Shear panels above the ground floor must have shear panels in the same bay and direction at every floor level below them.

Using the following guidance, the diagonal straps are sized to resist the total horizontal loads at each floor level as defined in Equations C-12 and C-13, based on trial shear panel locations and aspect ratios. Then the greater loads defined in Equations C-17 and C-18 are used to size the shear panel columns. Finally the panel connections and anchors are designed based on the guidance that follows.

[2] Specification for the Design of Cold-Formed Steel Structural Members, American Iron and Steel Institute (AISI), 1996.

7. DIAGONAL STRAP DESIGN. The diagonal straps are designed to resist the seismic story shears, V_x given in Equation C-27 that has been increased by the additional shear force due to torsion (Q_{si} in Equation 3.1). The shear panels shall be configured and diagonal straps sized so that the lateral shear panel design strength, $\phi_t Q_{sy}$ satisfies the following equation (see Equation C-34 in Appendix C).

$$\phi_t Q_{sy} = \phi_t \sum_{i=1}^{n} \left[n_s b_s t_s F_{sy} \left(\frac{W}{\sqrt{H^2 + W^2}} \right) \right] \geq V_x + Q_{si} \qquad \text{(Eq 3-4)}$$

Where:

ϕ_t = the resistance factor for tensile members (0.95),
n = the number of shear panels in the building frame for which the shear forces V_x and Q_{si} are applied.
n_s = the number of diagonal straps (panel faces with straps) in an individual panel.
F_{sy} = the design yield strength of the strap.

The number of shear panels, panel width, height, and strap size and strength shall be designed according to Equation 3-4 to meet minimum lateral yield capacity. All diagonal strap material must be ASTM A653 steel. Diagonal straps may not use re-rolled steel, because the re-rolling strain hardens the material, increasing material strength variability and reducing elongation (see USACERL Technical Report, Chapter 4 for a discussion of this concern).

8. COLUMN DESIGN – Structural Tubing or Built-up from Studs. The columns of the Panel A configuration are built-up with cold-formed steel studs. These studs must be oriented to form a closed cross-section as shown on the Test Panel A1 and A2 drawings in Appendix B. Individual studs must be welded to each other with a weld thickness equal to the thickness of the studs. The welds are intermittent, with a length and spacing that will ensure composite behavior of the column.

Structural tubing column design (Panel D configuration - Drawing D1 in Appendix B) follows the same procedure, but consists of a single member which is a closed section by itself. The equations in this guidance are used such that the number of studs making up this column is one.

a. Column Applied Loads. Loads applied to the columns are defined based on Equation C-17 (TI 809-04, Equations 4-2 and 4-6), where the effects of gravity load and seismic forces are additive and diagonal strap overstrength is accounted for. Only that portion of gravity loads applied to the tributary area of the shear panel columns are included in the design of these columns. However, the full horizontal seismic force, $\Omega_0 Q_E$ applied to the shear panel and resisted by the diagonal straps, will add a vertical component to the columns, increasing axial load. This horizontal load is based on the actual designed area of the diagonal straps as defined in Equation C-16. The total column axial load at the maximum ultimate stress in the diagonal straps, P_{vumax} is:

$$P_{vu\,max} = \frac{GL_{max}}{2} + F_{su\,max} n_s b_s t_s \left(\frac{H}{\sqrt{H^2 + W^2}} \right) \qquad \text{(Eq 3-5)}$$

Where:

GL_{max} = the maximum gravity load per shear panel, i.e., from $(1.2 + 0.2S_{DS})D + 0.5L + 0.2\,S$, in Equation C-17.
F_{sumax} = the maximum estimated ultimate stress in the diagonal straps, which equals to 1.5 F_{su} for ASTM A653 Grade 33 steel (F_{su} = 310 MPa and 45 ksi) and 1.25 F_{su} of Grade 50 steel (F_{su} = 448 MPa and 65 ksi).

b. Column Axial Capacity. Column capacity is determined based on AISI provisions. The design strength, P shall be determined based on the AISI guidance (Section C4 Concentrically Loaded

Compression Members). This guidance, applied to columns built-up with cold-formed steel studs or individual structural tubing members, is summarized in Appendix C (Paragraph C13). Columns shall be designed such that their design strength, P (Equation C-35) exceeds the total axial applied load, P_{vumax}.

c. Column Bending Load and Composite Behavior. The column anchor design provisions developed later in this guidance will create a moment connection. The primary purpose of the anchor design is to resist shear and uplift forces. However, this anchor design will also allow the columns to act as a moment frame, providing limited structural redundancy and widening of the hysteretic load deflection envelopes of the shear panel. This will allow the panels to absorb more energy under cyclic seismic loading conditions. The columns built-up from studs must be designed to act as a composite cross section in order to provide this moment capacity. This will require welding between the studs that will provide the shear transfer needed to develop the maximum moment in the columns. When one diagonal strap is in tension, the full gravity load on the shear panel may be carried in a single column, with the other column having no axial load. The maximum moment in a column will occur when it has no axial load. Therefore the welds shall be designed for the full moment capacity of the columns. This design requirement will allow the shear panel columns to continue providing bending resistance beyond the lateral yield deflection of the columns. These welds shall resist the maximum shear between the studs, which will be between the studs closest to the column neutral axis. This shear, q is defined as follows:

$$q = \frac{V_c Q}{I_c} \qquad \text{(Eq 3-6)}$$

Where:

V_c = the maximum column shear due to column moment only.

Q = the moment of the column cross-sectional area on one side of the critical weld about the critical weld plane.

I_c = the moment of inertia of the column due to bending in the plane of the shear panel.

The maximum column shear, V_c due to the maximum column moment M_c only is determined as follows:

$$V_c = \frac{2M_c}{H} = \frac{2F_{cy}I_c}{Hc} \qquad \text{(Eq 3-7)}$$

Where:

H = the panel height

F_{cy} = the yield strength of the column. This strength is not increased for column material overstrength because weld failure is controlled by the column material strength, so that any material overstrength would result in a proportionately greater weld strength.

c = the distance to the column neutral axis to the extreme fiber in the plane of the shear panel.

The moment of the column cross-sectional area on one side of the critical weld about the critical weld plane, Q is defined as follows:

$$Q = \int_A y\,dA = A\bar{y} \qquad \text{(Eq 3-8)}$$

Where:

A = the area of column cross-section on one side of the critical weld plane closest to the column neutral axis.

$\bar{y}$ = the distance from the neutral axis of the column cross-sectional area on one side of the critical weld plane to the critical weld failure plane.

Built-up columns are fabricated by welding individual studs together to form a closed cross-section, using flare V-groove welds. The same weld size and spacing shall be used between all studs in the built-up column. These welds are design according to AISI (Section E2.5 Flare Groove Welds), assuming double shear. The maximum spacing between centers of intermittent welds, s_{max} is determined as follows:

$$s_{max} = 1.5\phi_G t_c F_{cu} \frac{L}{q} \tag{Eq 3-9}$$

Where:

ϕ_G = the resistance factor for flare grove welds, equal to 0.55.
t_c = the stud thickness of the built-up columns.
F_{cu} = the ultimate strength of the column steel.
L = the length of intermittent grove welds.
q = the maximum shear determined in Equation 3-6.

Intermittent welds shall be made at both the top and bottom ends of the columns, regardless of the maximum center-to-center spacing, s_{max}.

d. Column Combined Axial and Moment Capacity. The combination of axial load and bending shall be evaluated using a modification to AISI guidance (C5.2.2 Combined Compressive Axial Load and Bending – LRFD Method). The combination of axial and moment on the column shall be evaluated based on the following interaction equation (modification of AISI Equation C5.2.2-2):

$$I = \frac{P_{vu\,max}}{F_{cy} A_c} + \frac{M_a}{M_{nx}} \le 1.0 \tag{Eq 3-10}$$

Where:

P_{vumax} = the applied axial load, defined in Equation 3-5.
A_c = the nominal column cross-sectional area.
M_a = the applied moment at maximum estimated strap yield strength, defined in Equation 3-11. This equation conservatively assumes the column is fully fixed at its top and bottom by the panel anchors. This moment is also conservatively based on the maximum panel lateral deflection at which the diagonal strap will yield. This moment includes the column bending and P-delta effect of axial load. Still, this moment will be less than the column moment with no axial load (paragraph 3-8c). The applied moment, M_a is defined as follows:

$$M_a = \frac{6EI_c \delta_{sy\,max}}{H^2} + P_{vu\,max}\delta_{sy\,max} \tag{Eq 3-11}$$

Where:

δ_{symax} = the maximum estimated lateral panel deflection at the maximum estimated yield strength of the diagonal straps, F_{symax} and is defined as follows:

$$\delta_{sy\,max} = \frac{F_{sy\,max}}{E}\left(\frac{H^2 + W^2}{W}\right) \tag{Eq 3-12}$$

Where:

F_{symax} = maximum estimated yield stress of the diagonal straps, equal to $2F_{sy}$ for Grade 33 and $1.5F_{sy}$ for Grade 50 steel.

M_{nx} = the column gross cross-section nominal moment capacity, and this is defined as follows (modification of AISI Equation C3.1.1-1):

$$M_{nx} = F_{cy}\left(\frac{I_c}{h_c - c}\right) \qquad \text{(Eq 3-13)}$$

Where:

h_c = the width of the column in the plane of the shear panel.
c = the distance from the column neutral axis to the extreme fiber.

e. Column Shear Capacity. The trial column design must be checked for shear capacity. The diagonal straps fasten to the columns near their connection to the tracks and column anchor. Therefore the column must either have adequate shear capacity for the maximum horizontal seismic force, $\Omega_0 Q_E$ applied to the shear panel, or the column shear capacity must be augmented with other components. The column shear design strength, V_C shall be determined based on AISI guidance (Section C.3.2, Strength for Shear Only). This guidance, applied to columns built-up with cold-formed steel studs or individual structural tubing members, is summarized in Appendix C (Paragraph C14). For columns built-up with studs, the maximum stud flange width over thickness, $(h/t)_{max}$ is defined as follows:

$$\left(\frac{h}{t_c}\right)_{max} = 0.96\sqrt{\frac{Ek_v}{F_{cy}}} \qquad \text{(Eq 3-14)}$$

Where:

h = the depth of the flat portion of the column web, which equals the stud flange width.
t_c= the column web thickness, which equals the stud thickness.
k_v = the shear buckling coefficient, which equals 5.34.

For studs with a flange width of 50-mm (2 inches) this requires a minimum stud thickness of 0.77-mm (30 mil or 20 gauge) for 33 ksi steel and 0.95 mm (37 mil or 18 gauge) for 50 ksi steel.

Columns have insufficient shear capacity by themselves, and require additional shear capacity from their anchorage detail (see paragraph 3.10a for anchorage shear design guidance).

9. CONNECTION DESIGN.

a. Connection Design Assumptions and Applied Loads. This paragraph provides connection design assumptions that define loading and load path issues for cold-formed steel shear panels. These assumptions apply to the diagonal strap-to-column connections. These loads are based only on the maximum lateral force, $\Omega_0 Q_E$. This force results from the right-hand term in Equation C-18, $\Omega_0 Q_E$, which accounts for diagonal strap material overstrength. The maximum estimated ultimate force in the diagonal straps (in the axis of the straps), P_{su}, is:

$$P_{su} = F_{su\,max} n_s b_s t_s \qquad \text{(Eq 3-15)}$$

The diagonal strap-to-column connection shall be designed to resist the force defined by Equation 3-15.

Panel design will require the use of angle section anchors as described under panel anchors (pargraph 10), because of the shear transfer requirements. This anchor will also transfer loads between the column and base or top beam, or floor slabs, thereby eliminating the need for load transfer with a column-to-track connection. In low seismic zones it may be possible to transfer the shear forces with a column-to-track connection only, without anchors. However, it is considered more reasonable to use fewer shear panels rather than many with low lateral load capacity. Therefore all shear panel design guidance presented here shall require the use of anchors. Anchor design is presented later in this document (Paragraph 10).

b. Screwed Fastener Connection Design. Self-tapping screwed connection capacity definition shall follow AISI guidance (Section E4 Screw Connections). Screws shall be installed and tightened in accordance with the manufacturer's recommendations. Screw connections loaded in shear can fail in one mode or in combination of several modes. These modes are screw shear, edge tearing, tilting and subsequent pullout of the screw, and bearing of the joined materials. The commentary of the AISI Specification (E4.3) gives further explanation and illustration of these modes of failure. The AISI provisions focus on the tilting and bearing modes of failure. Two cases are given depending on the ratio of the connected member thicknesses. Normally the head of the screw will be in contact with the thinner material, t_1. However, when both materials are the same thickness or the thicker member is in contact with the screw head, tilting becomes a more critical mode of failure. The AISI Section E4 guidance on design shear strength per screw, P_s applied to diagonal strap-to-column screw connections is summarized in Appendix C (Paragraph C15). The modes of failure expressed in Equations C-48 through C-52 are defined alongside the equations.

Minimum spacing guidance (AISI E.4.1) requires that the distance between centers of fasteners shall not be less than 3d, where d is the nominal screw diameter.

Minimum edge and end distance guidance (AISI E.4.2) requires that the distance from the center of a fastener to the edge of any connected part shall not be less than 3d. If the connection is subjected to shear force in one direction only, the minimum edge distance shall be 1.5d in the direction perpendicular to the force.

Finally, the minimum number of screws required at each diagonal strap-to-column connection, n_{screws} is calculated as follows:

$$n_{screws} \geq \frac{P_{su}}{n_s P_s} \qquad \text{(Eq 3-16)}$$

The nominal shear strength of the screws shall be determined based on manufacturer's data (AISI E4.3.2), which must be based on tests according to AISI Section F1(a). This nominal shear strength of approved screws must not be less than $1.25P_{ns}$ where P_{ns} is defined by Equations C-48 through C-52.

c. Design Rupture Strength. The design shear strength along a potential shear rupture plane between fasteners of connected members, V shall be determined as below (AISI E5 Shear Rupture, Specification and Commentary, AISC J4 Design Rupture Strength). The AISI commentary states that their shear rupture provisions are based on AISC provisions. The AISI provisions conservatively neglect the greater strength that AISC allows for the tensile rupture plane. The guidance below adds the greater strength allowed by AISC for this tensile rupture plane.

$$V = 0.6\phi_v F_u A_{nv} \qquad \text{(Eq 3-17)}$$

Where:

ϕ_v = the shear rupture resistance factor, equal to 0.75.
F_u = the ultimate strength of the member being evaluated.
A_{nv} = the net area subjected to shear along the rupture plane being considered.

The design tensile strength along a potential tensile rupture plane between fasteners of connected members, T shall be determined as follows:

$$T = \phi_t F_u A_{nt} \qquad \text{(Eq 3-18)}$$

Where:

ϕ_t = the tensile rupture resistance factor, equal to 0.75
A_{nt} = the net area subjected to tension along the rupture plane being considered.

The shear and tensile rupture strength are based on the diagonal strap ultimate strength of the member in the joint being evaluated. The maximum applied load on this joint is based on the yield strength of the same member, P_{sy}. This will be much less than the maximum estimated strap axial force, P_{su}. The maximum force in the members is not critical, but rather the minimum ratio of F_u/F_y because the rupture strength capacity is dependent on F_u and the maximum applied force is dependent on F_y. This guidance requires that ASTM A653 material be used for the straps and the minimum F_u/F_y ratio for Grade 33 and Grade 50 material is 1.36 and 1.30[3], respectively. These minimum ratios equate to yield, F_y and ultimate strengths, F_u of Grade 33 and Grade 50 material, such that F_{y33} = 228 MPa (33 ksi), F_{u33} = 310 MPa (45 ksi), F_{y50} = 345 MPa (50 ksi), and F_{u50} = 448 MPa (65 ksi). Therefore the strap yield strength, P_{sy} may be defined simply based on the yield strength of these materials. This requirement is expressed as follows:

$$(V + T)n_s \geq P_{sy} \qquad \text{(Eq 3-19)}$$

Where:

$$P_{sy} = F_y n_s b_s t_s \qquad \text{(Eq 3-20)}$$

When the strap-to-column rupture strength is evaluated based on Equation 3-19, the resistance factors in Equations 3-17 and 3-18 may be increased to 1.0, because of the ASTM minimum material requirement on F_u/F_y stated above.

d. Welded Connection Design. Welded design shall follow AISI guidance (Section E2 Welded Connections). This guidance covers connections of members in which the thinnest member is 0.18 inches or less. Arc welds shall be made in accordance with AWS D1.3[4] and its commentary. Resistance welds shall be made in accordance with the procedures in AWS C1.1 or AWS C1.3.

Welded diagonal strap-to-column connections will require fillet welds (AISI E2.4). The welds at the sides of the straps will be loaded in the longitudinal direction, and welds at the ends of the straps will be loaded in the transverse direction. The weld thickness should be equal to the thickness of the strap material. Ultimate failure of fillet welded joints has usually been found to occur by the tearing of the plate adjacent to the weld. The higher strength of the weld material prevents weld shear failure, therefore, this guidance is based on sheet tearing.[5] Fillet weld design for diagonal strap-to-column connections is summarized in Appendix C (Paragraph C16).

The fillet weld longitudinal and transverse shear strengths are based on the ultimate strength of the thinner member (normally diagonal strap) of the joint. The maximum applied load on this joint is based on the yield strength of the same member, P_{sy}. The maximum force in the members is not critical, but rather the minimum ratio of F_u/F_y because the rupture strength capacity is dependent on F_u and the maximum applied force is dependent on F_y. This guidance requires that ASTM A653 material be used for the straps and the minimum F_u/F_y ratio for Grade 33 and Grade 50 material is 1.36 and 1.30[6], respectively. Therefore, the strap yield strength, P_{sy} shall be defined simply based on the yield strength of these materials. This requirement is expressed as follows:

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[3] See AISI Dimensions and Properties for use with the 1996 AISI Cold-Formed Steel Specifications, ASTM Specifications for Referenced Steels.
[4] American Welding Society (1989), Structural Welding Code - Sheet Steel, ANSI/AWS D1.3-89, Miami, FL 1989.
[5] AISI Commentary, E2.4.
[6] See AISI Dimensions and Properties for use with the 1996 AISI Cold-Formed Steel Specifications, ASTM Specifications for Referenced Steels.

$$(P_L + P_T)n_s \geq P_{sy} \qquad \text{(Eq 3-21)}$$

10. PANEL ANCHORS. Panel anchors must be installed on both sides of the shear panel columns because the columns by themselves have inadequate shear capacity. Furthermore, if the column were simply fastened to the track, the track would be loaded in bending, due to uplift. The track is very weak in bending and this would violate the guidance stated in Paragraph 6c. Therefore, anchors consisting of angle iron sections shall be welded to both sides of the column at both the top and bottom of the columns to provide the required panel anchorage. Loose steel plates are laid over the horizontal portion of the angle sections. The angles and plates shall be drilled with through holes and anchored to the supporting diaphragm above and below the shear panel using embedded anchor bolts. See Appendix D (Figures D4 through D9) for examples of this anchor configuration.

a. Anchor Shear Capacity. Columns have insufficient shear capacity by themselves, and require additional shear capacity from their anchorage detail. This will require the installation of angle iron anchors on both sides of the columns, such that one leg of the angle extends beyond the critical shear plane. For screwed fastener connections, the critical shear plane is along the horizontal row of screws closest to the track in the diagonal strap-to-column connection. For the welded connections, the critical shear plane is along the strap-to-column weld near the track. The angle iron anchor shear design strength, V_A for a single angle is defined as follows:

$$V_A = 0.6\phi_v F_{Ay} b_c t_A \qquad \text{(Eq 3-22)}$$

Where:

$\phi_v = 1.0$.
F_{Ay} = the anchor angle iron yield strength.
b_c = the width of the angle, which equals the out-of-plane width of the column.
t_A = the thickness of the angle.

The total design shear strength, V_T must exceed the maximum shear panel horizontal seismic force P_{humax} ($\Omega_0 Q_E$). All anchors are made up with two angles, on either side of the column, so that V_T may be expressed as:

$$V_T = V_C + 2V_A \geq P_{hu\,max} \qquad \text{(Eq 3-23)}$$

Where:

V_C = the column shear capacity determined in according to Equation C-46 in Appendix C.

$$P_{hu\,max} = \Omega_0 Q_E = F_{su\,max} n_s b_s t_s \left(\frac{W}{\sqrt{H^2 + W^2}} \right) \qquad \text{(Eq 3-24)}$$

b. Anchor Angle and Plate Design. The most critical load condition for anchors is when the effects of gravity load and seismic forces counteract each other. This load condition is expressed by Equation C-18.

The selected angle and plate anchors shall resist the applied shear and uplift forces. These anchors will also provide limited moment resistance. The angles and plates will yield in flexure between the anchor bolts and bend in the angle, but will not fail in a brittle manner. This limited moment resistance will slightly widen the hysteretic envelope in the load deflection performance of the panel. The angles and plates can yield significantly through many cycles with no loss of shear and uplift resistance (some loss of moment resistance). The maximum weld thickness to the column shall be used, which is based on the thickness of the column material, as indicated in Table 3-3. The panel anchors shall be constructed using angle sections with a thickness equal to the maximum permitted based on the column-to-anchor weld thickness (see Table 3-4).

The limitation on angle thickness will cause the angle to yield in bending at the angle corner, so that it provides little resistance to uplift by itself. Uplift resistance shall be increased by adding a plate over the horizontal leg of the angle.

Table 3-3. Maximum Column-to-Anchor Weld Thickness.[7]	
Column Material Thickness, t_c	**Maximum Weld Thickness, t_w**
t_c < 6 mm (¼ inch)	$T_w = t_c$
$t_c \geq$ 6 mm (¼ inch)	$t_w = t_c$ – 1.5 mm (t_c – 1/16 inch)

Table 3-4. Maximum Angle Thickness Based on Column-to-Anchor Weld Thickness.[8]	
Weld Thickness, t_w	**Maximum Angle Thickness, t_A**
3 mm (1/8 inch)	6 mm (1/4 inch)
5 mm (3/16 inch)	13 mm (1/2 inch)
6 mm (1/4 inch)	19 mm (3/4 inch)
8 mm (5/16 inch)	29 mm (1-1/8 inch)[9]

The column-to-angle weld design strength, P_A shall exceed the total uplift force applied to one angle at one side of the column due to uplift and bending. This is expressed as follows:

$$\frac{P_{vy\,max}}{2} + P_M \leq P_A = P_T + P_G \qquad \text{(Eq 3-25)}$$

Where:

P_A = the total vertical design capacity of the column-to-angle weld

P_T = the design strength of the transverse loaded fillet weld at the horizontal column-to-angle weld (Equation C-56)

P_G = the design strength of the longitudinal loaded flare bevel grove weld at the vertical column-to-angle welds at the corner of the columns. The design strength for this column-to-angle weld shall be determined based on AISI guidance (Section E2.5 Flare Grove Welds). The application of this guidance to the design of column-to-angle welds is summarized in Appendix C (Paragraph C17).

P_{vymax} = the net anchor vertical load at the maximum yield stress in the diagonal straps, expressed by:

$$P_{vy\,max} = F_{sy\,max} n_s b_s t_s \left(\frac{H}{\sqrt{H^2 + W^2}} \right) - \frac{GL_{min}}{2} \qquad \text{(Eq 3-26)}$$

Where:

GL_{min} = the minimum gravity load per shear panel, i.e., $(0.9 - 0.2S_{DS})D$ in Equation C-18.

P_M = the uplift force capacity per anchor angle, beyond $P_{vymax}/2$ available to resist moment is determined by Equation 3-27. This assumes the anchor bolts are sufficiently tightened to provide a moment restraint.

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7 AISC Load and Resistance Factor Design (LRFD) Specification, 2nd Edition, 1994, Section J2b.

8 AISC LRFD, Table J2.4

9 Maximum thickness of standard angles.

$$P_M = \frac{M_A - \frac{P_{vy\max}}{2}\frac{d_b}{2}}{\frac{d_b}{2}} \qquad \text{(Eq 3-27)}$$

Where:

M_A = the plastic moment capacity of the angle and plate resting over the horizontal leg of the angle

d_b = the distance from the plate edge where the angle corner begins to the critical bending plane in the plate. The critical bending plane is at the edge of the anchor bolt nut(s) nearest to the columns.

The plastic moment capacity of the angle and plate, M_A is calculated as follows:

$$M_A = \phi_b F_{Ay} \frac{b_c}{4}\left(t_A^2 + t_p^2\right) \qquad \text{(Eq 3-28)}$$

Where:

ϕ_b = the bending resistance factor, equal to 0.90.
F_{Ay} = the yield strength of the angle and plate.
b_c = the length of the angle, which equals the anchor width and out-of-plane width of the column.
t_A = the thickness of the angle.
t_p = the thickness of the plate.

The distance from the plate edge to the critical bending plane, d_b is determined as follows:

$$d_b = d_c - k - \frac{W}{2} \qquad \text{(Eq 3-29)}$$

Where:

d_c = the distance from the center of anchor bolts to the column face.
k = the distance from the corner of the angle to the flat portion of the angle legs (from AISC LRFD, Dimensions and Properties of Structural Shapes).
W = the width across flats of the anchor bolt nut(s). This dimension is given in AISC LRFD, Volume II Connections, Table 8-2, Dimensions of High-Strength Fasteners.

The column moment connection capacity, M_c is defined as follows:

$$M_c = P_M(h_c + t_A + k) \qquad \text{(Eq 3-30)}$$

Where:

h_c = the depth or in-plane width of the column.

A portion of this moment is used to resist the moment created by the eccentric loading of diagonal strap-to-column connection with respect to the center of the column anchor, $P_{symax}L_s$. The angle uplift capacity that remains to resist column bending, P_{cb} shall be greater than zero and is determined as follows:

$$P_{cb} = \frac{M_c - P_{sy\max}L_s}{h_c + t_A + k} \qquad \text{(Eq 3-31)}$$

Where:

P_{symax} = the maximum yield strength of the diagonal strap(s) in the shear panel, in the axis of the strap. This is determined as follows:

$$P_{sy\,max} = F_{sy\,max} n_s b_s t_s \tag{Eq 3-32}$$

L_s = the diagonal strap eccentricity equal to the distance from the center of the diagonal strap-to-column connection to the center of the column-to-anchor connection, perpendicular to the axis of the diagonal strap.

c. Anchor Bolt Design. The anchor bolts that fasten the column anchors to the reinforced concrete beam or slab are next designed. The same detail used in the anchors at the base of the column shall be used in the anchor at the top of the column. The anchor bolts shall be sized based on the bolt shear strength, P_v tensile strength, P_t and cone failure design strength, P_c. The anchor bolt shear design strength, P_v shall exceed the applied shear load per bolt, P_{hAB}. This is expressed as follows:

$$P_v \geq P_{hAB} = \frac{P_{hu\,max}}{n_{AB}} \tag{Eq 3-33}$$

Where:

$$P_v = \phi_{tv} F_v \frac{\pi}{4} d_{AB}^2 \tag{Eq 3-34}$$

n_{AB} = the number of anchor bolts in the anchor on both sides of the column
ϕ_{tv} = the tensile and shear resistance factor (0.75[10]).
F_v = the nominal shear strength of the anchor bolts.[11]
d_{AB} = the diameter of the anchor bolt.
P_{humax} = the maximum shear panel horizontal force defined by Equation 3-24.

The anchor bolt-tensile design strength, P_t shall exceed the applied tensile force per bolt, P_{tAB}. The anchor bolt tensile strength, P_t[12] is determined as follows:

$$P_t = \phi_{tv} F_t \frac{\pi}{4} d_{AB}^2 \tag{Eq 3-35}$$

Where:

F_t = the nominal tensile strength of the anchor bolts determined by the minimum value given in AISC LRFD, Tables J3.2 (Design Strength of Fasteners) and J3.5 (Tension Stress Limit (F_t) for Fasteners in Bearing-type Connections. The value of f_v used in Table J3.5 is determined as follows:

$$f_v = \frac{P_{hAB}}{\frac{\pi}{4} d_{AB}^2} \tag{Eq 3-36}$$

[10] AISC LRFD, Table J3.2, Design Strength of Fasteners.
[11] AISC LRFD, Table J3.2, Design Strength of Fasteners.
[12] American Concrete Institute (ACI), Manual of Concrete Practice, Part 3, State-of-the-Art Report on Anchorage to Concrete – ACI 355.1R-91, 1991, equation 3.1.

The applied tensile force per anchor bolt, P_{tAB} is calculated as follows:

$$P_{tAB} = \frac{(P_{cb} + \frac{P_{sy\,max} L_s}{h_c + t_A + k} + \frac{P_{vy\,max}}{2})(W_A - \frac{t_A}{2})}{(W_A - d_c)(\frac{n_{AB}}{2})} \qquad \text{(Eq 3-37)}$$

The anchor bolt cone failure design strength, P_c shall exceed the applied tensile force per bolt, P_{tAB}. The anchor bolt cone failure design strength, P_c is determined based on the guidance in ACI 355.1R-91.[13] The applications of this guidance to anchor bolt design for shear panels is summarized in Appendix C (Paragraph C18). Appendix C (Paragraph C18) also defines the minimum edge distance for anchor bolts based on ACE 355.1R-91.

[13] ACI 355.1R-91, Equation 3.2.

CHAPTER 4
MASONRY VENEER/STEEL STUD WALLS
(NONBEARING CONSTRUCTION)

1. INTRODUCTION. This document defines the criteria to be used when designing curtain wall masonry veneer on steel studs. These walls are to be designed to resist out-of-plane lateral loads due to wind and seismic forces. Also, the wall system must collect, direct and remove water from the wall cavity to properly control moisture, prevent efflorescence, and control corrosion of the steel system. These curtain wall steel stud systems will not carry building dead or live loads, nor provide lateral resistance to the building system.

2. GENERAL DESCRIPTION OF WALL SYSTEM. This wall system consists of a masonry veneer exterior wythe connected by anchors to a steel stud backup wall. The steel studs will be mechanically braced until the sheathing and wallboard are placed on the studs. Sheathing is placed on the cavity face of the stud and the wallboard material is placed on the inside face of the studs. A cavity space is provided between the masonry veneer and the steel stud wall to allow moisture to migrate down the inside face of the cavity, brace the masonry veneer laterally and transfer the horizontally applied loads to the steel studs. The masonry wythe will be isolated on three sides to assure that it will only carry its own weight.

3. REQUIREMENTS FOR WALL COMPONENTS AND DETAILS. See Appendix G for typical Masonry Veneer / Steel Stud details.

a. Masonry Wythe.

(1) Masonry Units. Dimensional and physical requirements of the masonry units are given in TM 5-809-3/NAVFAC DM-2.09/AFM 88-3, Chapter 3. Masonry units used in veneer walls will be solid.

(2) Mortar. Four types of mortar are specified in ASTM C270, they are M, S, N, and O. While Types S and N may be used for masonry veneer systems; Type S mortar has higher strength and good workability and can be used above and below grade, Type N has a lower strength, better workability is more water tight and can only be used above grade. The lower strength of Type N mortar allows cracking in the masonry wythe at relatively low load levels. Masonry cracking will result in more uniformly distributed anchor forces. Conversely, vertical beam action of the uncracked masonry wythe causes nonuniform distribution of loads to the wall anchors. Those wall anchors near the top of the uncracked masonry wythe have much higher loads than those in the lower half of the wythe. Type S mortar is recommended for the design of masonry veneer steel stud systems. Two strength properties of mortars are measurable: the bond strength in accordance with ASTM C 1072 and the compressive strength in accordance with ASTM C 780. Of these two strengths bond is more important in veneer walls and compression is more important in bearing walls. Type S mortar has the highest compressive and bond strengths and can be achieved with either portland cement-lime or masonry cement. However, since masonry cements include premixed workability and air-content additives the bond strengths are reduced. In order to achieve good bond strengths the maximum air-content of the cement must be limited to 12 %. Since every manufacturer of masonry cement uses different additives and a different air-content, care should be used when specifying masonry cements for veneer walls. To assure a bond strength comparable to a portland cement lime mortar, a comparative bond strength test between the masonry cement proposed for the job and a corresponding type of portland cement mortar needs to be completed to demonstrate equivalent or better bond and compressive strengths for the masonry cement mortars. The contractor will be required to perform the ASTM C 1072 and C 780 tests on their proposed mortars. Masonry units for the bond strength test will be the same as proposed for the project.

(3) Masonry Base Details. The base of the masonry wythe must be placed on a shelf angle or a foundation ledge that is lower than the base channel of the steel stud wall by at least 102 mm (4 "). The width of this shelf angle or a foundation ledge will include the width of the masonry wythe and the cavity. This width will not be less than two thirds of the unit thickness plus the minimum air space. Masonry units set on shelf angles may use a formed lip to reduce the depth of the horizontal joint that is created at the shelf angle line.

b. Steel Studs and Framing. Designers specifying cold-formed studs and framing will use a minimum base metal thickness of 1.21 mm (0.0478 ") and not refer to the metal thickness as 1.146 mm (0.0451"). While the base metal thickness is to be specified to match the design, designers can use table 1-1 as a guide to selecting commonly referred base metal thickness. Designers should be aware that the minimum delivered thickness specified for steel studs will be no less than shown in table 1-1 when materials are specified to the minimum design thickness and delivered in accordance with AISI. The minimum depth of members will be 89 mm (3-1/2 ") and the minimum flange width of 35 mm (1-3/8 ") will have a minimum return lip of 6.4 mm (1/4 "). Shop drawing submittals will need to present the calculations that show the effective flange, with the return lip provided. The actual required stud depth, thickness and spacing will be determined prior to completion of the contract documents. In some cases, the use of a minimum stud depth of 152 mm (6") yield a more efficient design. Steel studs and framing will be hot-dipped galvanized metal with a minimum ASTM A 653, G60 coating.

(1) Welding. Welding of steel studs requires the use of qualified welders experienced in the welding of cold-formed steel. When welding is used, the contractor needs to provide Qualification documents for each welder working on the project. Welded connections to steel framing members will be touched-up with zinc-rich paint after welding. Normally welded connections are not required in curtain-wall construction but may be used for attachments.

(2) Connections. Connections of studs to runners and other framing members will be made with screws or welds. When the thickness of the thinner connected parts is less than 4.8 mm (3/16 ") the capacity of the connected parts will be in accordance with AISI. For bolted connections when the thickness of the thinner material is equal to or greater than 4.8 mm (3/16 ") use AISC. Normally the minimum top and bottom channel connections in curtain-wall construction will require a single #10-16 self-drilling, self-tapping screw in each flange of the bottom runner for either system and the top channel of the double track system. Welding of studs may be used in lieu of screws or bolts. Slide clip connections will be used when parapets extend above the roof line of the structure. These slide clips connectors need to be welded to a structural element and the details and the capacity of the manufacturers system will need to be included in the shop drawing review. The minimum edge distance of shot-in anchors for top and bottom runners to concrete is 76 mm (3 ") and the minimum fastener spacing is 102 mm (4 "). Contract plans will show anchorage details of the steel studs and other framing members to the building structural system, and the extra steel stud wall framing members required around openings. The required strength capacities of framing weld and screw connectors will be in accordance with ML/SFA 540-87 and ML/SFA 920-91.

(3) Openings. Window and door frames will be attached to the steel stud system, not the masonry veneer.

(4) Top Track and Bottom Runner. A single track or double track top (slip joint) connection will be used at the top of stud walls to permit the vertical movement of the structural framing system to prevent the loading of the steel studs. Both flanges of the steel stud will be attached to the inside top channel for the double track system and the bottom runner of either system. Mechanical bracing of the single track systems should

be installed in addition to support for all studs. This brace will be placed 305 to 457 mm (12 to 18 “) from the bottom of the top channel track.

(5) Parapets. When parapet walls extend above the roof line a slip clip connection will be used to allow for structural deflections without loading the steel stud system.

c. Sheathing. Fire and eater resistant gypsum board sheathing encased in water-repellent paper on both sides and on the long edges will meet the requirements of ASTM C 79. Other materials may be used as sheathing when supported by satisfactory performance data. All gaps in the sheathing resulting from cuts, corners, joints and machine end cuts of the sheathing and all joints at the interface of the sheathing with dissimilar materials such as floor slabs, doors, windows, and other locations where the water-resistant membrane terminates will be taped or filled with an exterior rubber-based caulk. The base detail of the exterior sheathing will be designed to resist water infiltration and the caulking will be applied to form joints that are complete and continuous. Enough connections of the sheathing to the steel studs to provide lateral support for the studs in the direction parallel to the plane of the wall will be required. All screw attachments through the exterior sheathing must resist air and water infiltration.

d. Veneer Anchors. Veneer anchors will be attached through the sheathing to the steel studs. All steel components of anchors will be stainless steel or hot-dipped galvanized steel. Anchor wires will be a minimum of 4.8 mm (3/16 “) diameter. There will be one anchor for each 0.19 m^2 (2 ft^2) of wall area and anchors will be spaced no further apart than 610 mm (24 “). The load-deflection stiffness criteria of each veneer anchor, applies to direct loads in both tension and compression, and will be not less than 350 N/mm (2000 lbs/in). The design load of the anchor will be the controlling wind load on the stud tributary width times one-half the vertical span of the stud. The controlling wind load will be the lesser of the design wind load or the wind load that causes masonry cracking. This load will then be used to calculate the required anchor capacity. Anchors will have a maximum “play” or not more than 1.59 mm (1/16 “). Synthetic rubber washers will be used under tie connector plates. A clutch torque slip screw gun will be used to eliminate stripping of threads. Additional anchors will be installed within 305 mm (12 “) of the free edges of veneer panels and at the edges of wall openings at the normal spacing. Additional anchors required around openings will be detailed on the contract drawings.

e. Fasteners. Screw connectors for stainless steel anchors will be stainless steel. Screws for galvanized anchors will be hot-dipped galvanized.

f. Moisture Barrier. A water-resistant membrane will be placed over the exterior sheathing. The membrane will be 67 N (15 lbs) roofing felt or similar material, such as: “Tyvek” building wrap by Dupont or “Barricade” building wrap by Simplex. The barrier material will be shingled with each sheet lapping over the sheet below with a minimum 152 mm (6”) lap. Sheets will be lapped a minimum of 152 mm (6”) at vertical joints. The moisture barrier must not be a vapor barrier that will trap water in the stud space of the wall.

g. Vapor Retarder. A 0.15 mm (6-mil) vapor retarder as required in the ROUGH CARPENTRY guide specification will be provided on the warm side of the insulation. In most geographical areas the vapor retarder will be located between the interior wall board material and the face of the steel studs. In hot humid areas of the United States including the Gulf coast, Florida, coastal Georgia, North Carolina, South Carolina and Virginia, the vapor retarder should be located between the exterior sheathing and the steel studs. Check local practice and the recommendations for the installation of insulation and vapor retarders at the project location. The building designers should perform a condensation analysis of the wall system used to determine the dew point location within the wall were condensation might be expected occur.

h. Flashing. Copper or stainless steel through-wall flashing as required in the SHEET METALWORK, General guide specification will be used. Flashing must be designed, detailed and constructed so that all water entering the cavity is directed out through weep holes. Ends and sill flashing must be lapped and sealed at joints. Ends will be turned up at sills and heads. Flashing must also be turned up behind the moisture barrier a minimum of 152 mm (6") and will be attached to the sheathing. Flashing must extend to the exterior face of the masonry wall. Weep holes as described herein will be provided.

i. Shelf Angles. Shelf angles will be hot-dipped galvanized structural steel members. Angles will be provided in segments, approximately 3.1 m (10 ') in length, with gaps between segments. Shelf angles will be detailed to allow enough gaps for thermal expansion and contraction of the steel in angle runs and at building corners. Corners of buildings will have corner pieces with each leg no less than four 1.2 m (4 ') in length where possible. Any areas that are welded will be touched-up with a zinc-rich paint.

j. Cavity. A cavity space of 51 to 102 mm (2 to 4 ") will be provided between the masonry veneer and the exterior sheathing or, if insulation is used over the sheathing, between the masonry veneer and the insulation. In all situations a 51 mm (2 ") minimum wide air space is required and needs to be coordinated with the standard dimensions of lintels and shelf angles. The cavity provides water drainage and prevents moisture migration from the masonry wythe to the steel stud backup wall. The cavity should be kept clean of mortar droppings. To keep mortar droppings from plugging the weep holes place a course gravel or drainage material behind the weep holes in the cavity to a minimum depth of 102 mm (4").

k. Masonry Crack Control. Crack control will be in accordance with the Masonry Manual for anchored veneer.

l. Weep Holes. Head joint weep holes that extend through the masonry wythe will be provided immediately above the mortar bed joint containing the horizontal leg of the through wall metal flashing and near the top of the wall at the same spacing. Details along with the required spacing will be shown on a wall section on the contract drawings. Weep holes need to be kept free of debris during construction and need to be functional at the end of the construction period.

m. Head Joint Vents. Head joint vents will be placed near the top of the veneer wythe at the same spacing as the weep holes. These vents will help maintain a dry cavity.

4. WALL SYSTEM DESIGN REQUIREMENTS. This exterior wall system will be designed assuming that all out-of-plane lateral loads are resisted entirely by the steel stud backup wall. The veneer anchors will be designed to transfer those lateral loads to the steel studs. All in-plane loads will be isolated from the stud wall system. All vertical masonry loads will be carried by a shelf angle or the foundation wall. Veneer anchorage will provide sufficient movement to account for the story drift displacements around window and door openings.

a. Steel Studs. Studs will be sized and spaced to resist wind or seismic loads. Wind loads on steel studs and framing will be in accordance with EI 01S901 and seismic loads will be in accordance with TI 809-04. Supplemental framing will be added at the heads, jambs and sills of openings, as required by design, to resist the tributary loads from the opening closures (doors, windows, etc.). The stud system selected for lateral loading will be checked for deflection, which normally controls panel wall design. Material thickness for the top and bottom runners will be designed for allowable stress and deflection and will be equal to or thicker than the steel stud used in the wall. The minimum delivered material thickness for steel studs is shown in table 1-1.

(1) Section properties. All section properties needed for the design of the steel studs and framing will be in accordance with AISI.

(2) Allowable stresses. All allowable stresses used for the design of the steel studs and framing will be in accordance with AISI.

(3) Deflection limits. The allowable steel stud horizontal deflection, Δ due to the controlling lateral load is defined as follows:

$$\Delta \leq \frac{L}{600} \qquad \text{(Eq 4-1)}$$

Where:

L = The vertical span of the steel studs in mm (in).

(4) Selection of top and bottom runners. The base metal thickness for the steel stud runners is equal to or greater then for the steel stud. The thickness of the top runner, t will be sized as follows:

$$t = \left[\frac{7.5338(P)(e)}{F_y(b_{eff})}\right]^{1/2} \qquad \text{(Eq 4-2)}$$

Where:

P = the top end wind load reaction (lbs).
e = the gap between the inner top track and the outer top track as shown in figure 4-1, (in).
F_y = the yield strength of the top runner metal (psi).
b_{eff} = the effective width of the top channel flange for analysis (in).

For double track systems, b_{eff} is equal to the stud spacing. With single track systems, b_{eff} is equal to:

$$b_{eff} = W_{stud} + 2\left[\frac{e + D}{\tan(30)}\right] \qquad \text{(Eq 4-3)}$$

Where:

W_{stud} = the width of the stud flange (in).

D = depth of track overlap in a slip track connection, 32 mm (1.25 in).

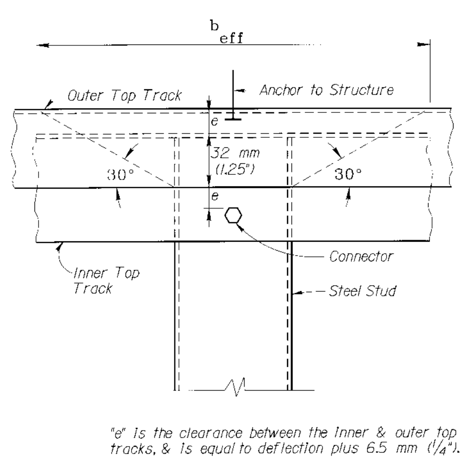

Figure 4-1: Double Track Slip Joint

b. Veneer Anchors. Anchors will be structurally adequate to transfer the lateral loads to the steel stud wall in both tension and compression. Capacity will be based on test results of the anchor provided. The following design procedure to determine the required anchor capacity.

Step 1: Calculate the maximum static out-of-plane distributed load at first cracking.

$$W_{cr} = C1\left[\frac{t_n}{L}\right]^2 \qquad \text{(Eq 4-4)}$$

Where:

W_{cr} = the cracking distributed load for the masonry KPa (psf),
t_n = the nominal thickness of the masonry wall mm (in),
L = the stud span m (ft),
C1 = constant =0.001644 – metric, (240 – English).

Step 2: Calculate the total static out-of-plane distributed load corresponding to the cracking wind load.

$$W_{tot} = W_{cr}\left[\frac{E_b I_b + E_s I_s}{E_b I_b}\right] \qquad \text{(Eq 4-5)}$$

Where:

W_{tot} = the total static out-of-plane wind distributed load corresponding to the cracking wind load KPa (psf),
$E_b I_b$ =the rigidity of the masonry veneer for the stud spacing N-mm^2 (Kip-in^2),
$E_s I_s$ = the rigidity of a steel stud N-mm^2 (Kip-in^2).

Step 3: Compare W_{tot} to W_d (the design wind load) to calculate the maximum anchor force (A_{max}).

$$\text{if} \ldots W_{tot} \leq W_d \ldots \text{then} \ldots A_{max} = \frac{W_{cr} L}{2}\left[\frac{\text{Stud .Spacing}}{C2}\right] \qquad \text{(Eq 4-6)}$$

or

$$\text{if} \ldots W_{tot} \rangle W_d \ldots \text{then} \ldots A_{max} = \frac{W_d L}{2}\left[\frac{E_b I_b}{E_b I_b + E_s I_s}\right]\left[\frac{\text{Stud.Spacing}}{C2}\right] \qquad \text{(Eq 4-7)}$$

Where:

Stud Spacing is in mm (in)
C2 = constant = 1000 – metric, (12 – English)

Step 4: Compare A_{max} to the seismic criteria.

$$\text{if} \ldots A_{max} \geq A_{seis} \ldots \text{then} \ldots A_{des} = A_{max} \qquad \text{(Eq 4-8)}$$

$$\text{if} \ldots A_{max} \langle A_{seis} \ldots \text{then} \ldots A_{des} = A_{seis} \qquad \text{(Eq 4-9)}$$

$$A_{seis} = C3(S_{DS})(V_w A_r) \qquad \text{(Eq 4-10)}$$

Where:

A_{max} = the maximum anchor load N (lbs),
A_{des} = the design wind anchor load N (lbs),
A_{seis} = the seismic anchor load N (lbs),
V_w = the veneer unit weight KPa (psf),
A_r = the area per anchor m^2 (ft^2),
C3 = constant =2,000,000 – metric, (2 – English).
S_{DS} = the design spectral response acceleration as per Eq C-3

Step 5: Choose the anchor capacity.

if K<= 350 N/mm (2 kips/in) then A_{cap} = 1.25 A_{des} (Eq 4-11)
if K> 350 mm (2 Kips/in) then A_{cap} = 2 A_{des} (Eq 4-12)

Where:

K = the anchor stiffness with the load in N/mm (kips/in) of deflection,
A_{cap} = the specified load capacity of the anchor to be selected N (lbs).

This design procedure was derived from Western States Clay Products Association test data.

c. Shelf Angles. The angle size and the attachment details of the angle to the building structure to provide vertical support of the masonry wythe must be determined by the designer. The deflection of the shelf angle under gravity loading due to the masonry will be limited to not more than 1.6 mm (1/16 “) at the end of the horizontal leg. Rotation of the shelf angle support will be included in the 1.6 mm (1/16 ”) deflection limit for the horizontal leg displacement calculation.

5. WORKMANSHIP. The success of this wall system is dependent upon good detailing and good workmanship. Careful, quality workmanship is essential to produce a satisfactory masonry veneer steel stud wall system. Special attention needs to be given to the masonry veneer construction to provide quality mortar, full head joints, bed joints, clean cavities, proper flashing and weep holes, and to provide mortar free brick expansion joints. It is essential that the design and detailing requirements in this document and references contained herein are clearly provided in contract plans and specifications.

6. DESIGN EXAMPLE.

a. Given:

(1) Wall System: An exterior non-loadbearing brick veneer steel stud wall system is required. The wall section consists of a nominal 102 mm (4 “) face brick wythe, a 60 mm (2-3/8 “) cavity, and a backup wall of 152 mm (6 “) steel studs with a 13 mm (1/2“) gypsum sheathing fastened to each side of the studs. The wall has a vertical span of 3.66 m (12 ‘) and is simply supported.

(2) Design lateral Loading: The design wind pressure for the wall is 1.34KPa (28 psf) acting inward or outward on the wall. The structure is in seismic zone zero.

b. Problem: Check the design of the wall system described above. Check the allowable flexural stress and the allowable deflection of the steel studs due to that loading.

c. Solution:

(1) Member loading requirements: Since the steel studs are to be designed to resist the total lateral (wind) loading, the anchors need to be selected to transfer the lateral loading from the brick wythe to the steel studs.

(2) Assumptions:

(a) Studs: The minimum stud to be checked is: a 152 mm (6 “) stud; 1.438 mm (0.0566 “) thickness, 41 mm (1-5/8 “) flange width and with a 13 mm (1/2 “) return. Studs will be spaced on 406 mm (16 “) centers.

(b) Anchors: The brick wythe is attached to the steel studs backup wall with 4.8 mm (3/16 “) diameter corrosion resisting veneer anchors spaced 406 mm (16 “) on center both vertically and horizontally.

(3) Check stud strength:

(a) Section Properties: From the American Iron and Steel Institute (AISI, Specification for the Design of Cold-Formed Steel Structural Members, Part V, “Charts and Tables”.

Thickness of sheet (t) = 1.438 mm (0.0566 “),
Depth of section (D_{stud}) = 152 mm (6.00 “),
Width of flange (B) = 41 mm (1.625 “),
Return of flange (C) = 15 mm (0.60 “),
Section Modulus (S_x) = 17,649 mm^3 (1.077 in^3),
Moment of inertia (I_x) = 1,318,205 mm^4 (3.167 in^4),
Area of section (A_{stud}) = 15.6 mm (0.616 “).

(b) Steel Properties:

Modulus of elasticity (E_s) = 200,000 MPa (29,000,000 psi),
Yield strength (F_y) = 228 MPa (33,000 psi).

(c) Design check: The uniform wind load normal to the wall, w, in N/m (lbs/ft) of wall, is:

$$w = (1.34\text{KPa})\left[\frac{406\text{mm}}{C4}\right] = 544\text{N/mm}...(37.3\text{plf}) \qquad \text{(Eq 4-13)}$$

Where:
C4 = constant = 1 - metric, (12 – English)

The moment in steel stud due to wind load, M_s, is:

$$M_s = \frac{wL^2}{8} = \frac{544\ \text{N/m}..(3.66\,\text{m})^2}{8} = 911\ .\text{N}-\text{m}...(672\ \text{ft}-\text{lbs}) \qquad \text{(Eq 4-14)}$$

Where:

L = the span of the steel studs in m (ft).

The allowable moment in steel stud with a 1/3 increase is stress for wind, M_a, is:

$$M_a = \frac{1.33F_yS_x}{\Omega_f} = \frac{1.33(228MPa)(17{,}649mm^3)}{1.67(C5)} = 3{,}198.N - m...(2{,}359ft - lb) \quad \text{(Eq 4-15)}$$

Where:

Ω_f = the required factor of safety for bending ad given in the AISI specification,
C5 = constant =1,000,000 – metric, (12 – English).

$$M_a = 3{,}198.N - m..(2{,}359ft - lbs)..\rangle\rangle..M_s 911.N - m..(672ft - lbs) \quad \text{(Eq 4-16)}$$

(4) Check stud deflection: The deflection, Δ, in the steel stud due to the wind load is:

$$\Delta = \frac{5wL^4C6}{384E_sI_x} \quad \text{(Eq 4-17)}$$

$$\Delta = \frac{5(544.N/m)(3.66.m)^4(10^9)}{384(200{,}000MPa)(1{,}318{,}204.mm^4)} = 4.8.mm...(0.189\,in) \quad \text{(Eq 4-18)}$$

Where:

C6 = constant = 10^9 – metric, (1728 – English)

The allowable deflection in the steel stud, is given by:

$$\Delta \le \frac{L}{600} = \frac{3.66.m(C7)}{600} = 6.1.mm...(0.240in) \quad \text{(Eq 4-19)}$$

$$4.8.mm...(0.189\,in)\ldots\langle\ldots 6.1.mm...(0.240\,in)\ldots OK \quad \text{(Eq 4-20)}$$

d. Summary:

(1) A 152 mm (6 “), 1.438 mm (0.0566 ”) thick steel stud with a 41 mm (1-5/8 “) flange and a 15 mm (0.6 “) return, spaced at 406 mm (16 “) centers is satisfactory.

(2) Use a 4.8 mm (3/16 “) diameter wire anchor spaced on 406 mm (16 “) centers vertically and horizontally.

APPENDIX A

REFERENCES

GOVERNMENT PUBLICATIONS:

Department of the Army

TI 809-04, “Seismic Design of Buildings”

TI 809-01, “Load Assumptions for Buildings”.

TM 5-809-2, “Structural Design Criteria for Buildings”

TM 5-809-3/NAVFAC DM-2.09/AFM 88-3, Chapter 3. “Masonry Structural Design for Buildings.”

TM 5-809-10/NAVFAC P-355/AFM 88-3, Chapter 13. “Seismic Design for Buildings.”

CEGS 04255, “Nonbearing Masonry Veneer/Steel Stud Walls”

CEGS 05400, “Cold-Formed Steel Framing”

USACERL Technical Report, http://owww.cecer.army.mil/techreports/wilcfstr.post.pdf
Development of Cold-Formed Steel Seismic Design Guidance

USACERL Design Spreadsheet, http://owww.cecer.army.mil/techreports/wilcfsxl.post.pdf
Development of Cold-Formed Steel Seismic Design Guidance

WES-IM: CAD Libraries:

Standard Cold-Formed Steel Details

Masonry Veneer / Steel Stud Details

Federal Emergency Management Agency

FEMA 273: NEHRP Guidelines for the Seismic Rehabilitation of Buildings

FEMA 274: NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings

FEMA 302: NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part 1 - Provisions

FEMA 303: NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part 2 - Commentary

NONGOVERNMENT PUBLICATIONS:

American Institute of Steel Construction (AISC)
1 East Wacker Drive, Suite 3100, Chicago, IL 60601

Steel Construction Manual

American Iron and Steel Institute (AISI)
1101 17th Street, NW
Washington, DC 20036-4700

Specification for the Design of Cold-Formed Steel Structural Members

Cold-Formed Steel Design Manual (Parts: I through VI)

The Design and Fabrication of Cold-Formed Steel Structures

Lightweight Steel Framing Design Manual

Prescriptive Method for Residential Cold-Formed Steel Framing

Commentary on Prescriptive Method for Residential Cold-Formed Steel Framing

Fire-Resistance Ratings of Load-Bearing Steel Stud Walls

Corrosion Protection of Steel Framing Members

RG-9405: Thermal Design Guide for Exterior Walls

RG-9518: Design Guide for Cold-Formed Steel Trusses

RG-9604: Shear Wall Design Guide

AISI Report CF 93-1, Preliminary Design Guide for Cold-Formed C and Z Members

RG-933: Fasteners for Residential Steel Framing

RG-934: Low-Rise Residential Construction Details

Sixth Specialty Conference on Cold-Formed Steel Structures, Effective Lengths for Laterally Unbraced Compressions Flanges of Continuous Beams Near Intermediate Supports

American Society of Civil Engineers (ASCE)
1801 Alexander Bell Drive
Reston, Virginia 20191-4400

ASCE 7-95: Minimum Design Loads for Buildings and Other Structures

American Society for Testing and Materials (ASTM)
1916 Race Street, Philadelphia, PA 19103

A 370: Standard Test Methods and Definitions for Mechanical testing of Steel Products

A 500: Cold-Formed Welded and Seamless Carbon Steel Structural tubing in Rounds and Shapes

A 653/A 653M: Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process

A 792/792M: Steel Sheet, 55% Aluminum-Zinc Alloy-Coated by the Hot-Dip Process

A 924/A 924M: General Requirements for Steel Sheet, Metallic-Coated by the Hot-Dip Process

C 79: Treated Core and Nontreated Core Gypsum Sheathing Board

C 270: Mortar for Unit Masonry

C 645: Non-Load Bearing (Axial) Steel Studs, Runners (Tracks), and Rigid Furring Channels for Screw Application of Gypsum Board

C 754: Installation of Steel Framing Members to Receive Screw-Attached Gypsum Board

C 780: Standard Test Method for Preconstruction and Construction Evaluation of Mortars for Plain and Reinforced Unit Masonry

C 840: Application and Finishing of Gypsum Board

C 841: Installation of Interior Lathing and Furring

C 842: Application of Interior Gypsum Plaster

C 847: Metal Lath

C 926: Application of Portland Cement-Based Plaster

C 954: Steel Drill Screws for the Application of Gypsum Board or Metal Plaster Bases to Steel Studs From 0.033 in. (0.84mm) to 0.112 in. (2.84 mm) in Thickness

C 955: Load-Bearing (Transverse and Axial) Steel Studs, Runners Tracks), and Bracing or Bridging for Screw Application of Gypsum Board and Metal Plaster Bases

C 1002: Steel Drill Screws for the Application of Gypsum Board or Metal Plaster Bases

C 1007: Installation of Load Bearing (Transverse and Axial) Steel Studs and Related Accessories

C 1072: Standard Test Method for Measurement of Masonry Flexural Bond Strength

American Welding Society (AWS)
2501 N.W. 7th Street, Miami, FL 33125

C1.1: Recommended Practice for Resistance Welding

C1.3: Recommended Practice for Resistance Welding, Coated Low Carbon Steels

D1.3: Structural Welding Code - Sheet Steel

Canadian Forestry Service
Ottawa Ontario, Canada

Onysko, , Serviceability Criteria for Residential Floors Based on a Field Study of Consumer Response, Fortintek canada Corp. Report 03-05-10-008

Onysko, Some Background on Factors Affecting Performance of Floors and Setting of Performance Criteria, Task Grou Meeting Fortinek Canada Corp.

Center for Cold-Formed Steel Structures (CCFSS)
University of Missouri - Rolla
Department of Civil Engineering
Rolla, Missouri 65401

Cold-Formed Steel Design, Computer Programs

Report on Behavior of Cold-Formed Steel Sections with Web Openings

Technical Library, http://www.umr.edu/~ccfss/

Council of American Building Officials (CABO)
5203 Leesburg Pike, Suite 708, Falls Church, VA 22041

CABO: One and Two Family Dwelling Code

Gypsum Association
810 First Street, NE
Washington, DC 20002

Fire Design Manual
National Association of Architectural Metal Manufacturers (NAAMM)
8 South Michigan Avenue
Chicago, Illinois 60603

NAAMM HMMA 803-97: Steel Tables

NAAMM Standard ML/SFA 540-87: “Lightweight Steel Framing Systems Manual”, Third Edition, The National Association of Architectural Metal Manufacturers.

NAAMM Standard ML/SFA 920-91: “Guide Specifications for Metal Lathing and Furring”, Fourth Edition, The National Association of Architectural Metal Manufacturers.

Structural Stability Research Council
Lehigh University
Bethlehem, PA 18015

Yura, “Fundamentals of Beam Bracing”, Is Your Structure Suitably Braced?, April 1993

Virginia Tech University
Blacksburg, Virginia

Kraus, Murray, Floor Vibration Design Criterion for Cold-Formed C-Shaped Supported Residential Floor systems, Masters Thesis, 10 February 1997.

Western States Clay Products
San Francisco, California

KPFF, Report on Behavior and Design of Anchored Brick Veneer / Metal Stud Systems, September 1989.

APPENDIX B

COLD-FORMED STEEL TEST PANEL DRAWINGS

This Appendix shows a typical three story barracks framing layout, and the three panels as tested by CERL. The elevation views are a good representation of the typical shear wall panel layout. However, the connection details have been modified since the completion of testing. Designers are to use the new joint details as shown in the design example in appendix D when designing.

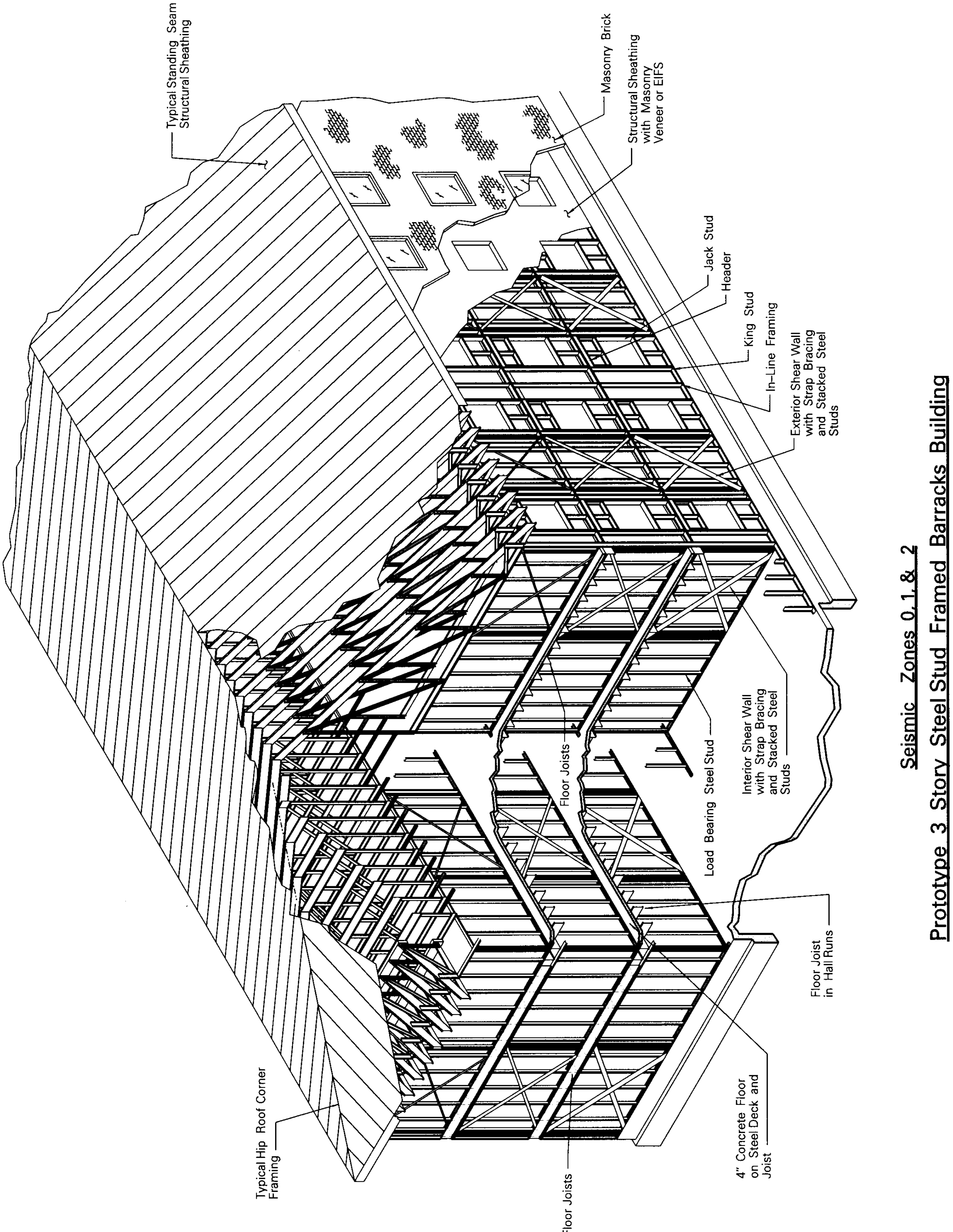

Figure B-1: Prototype 3 Story Barracks

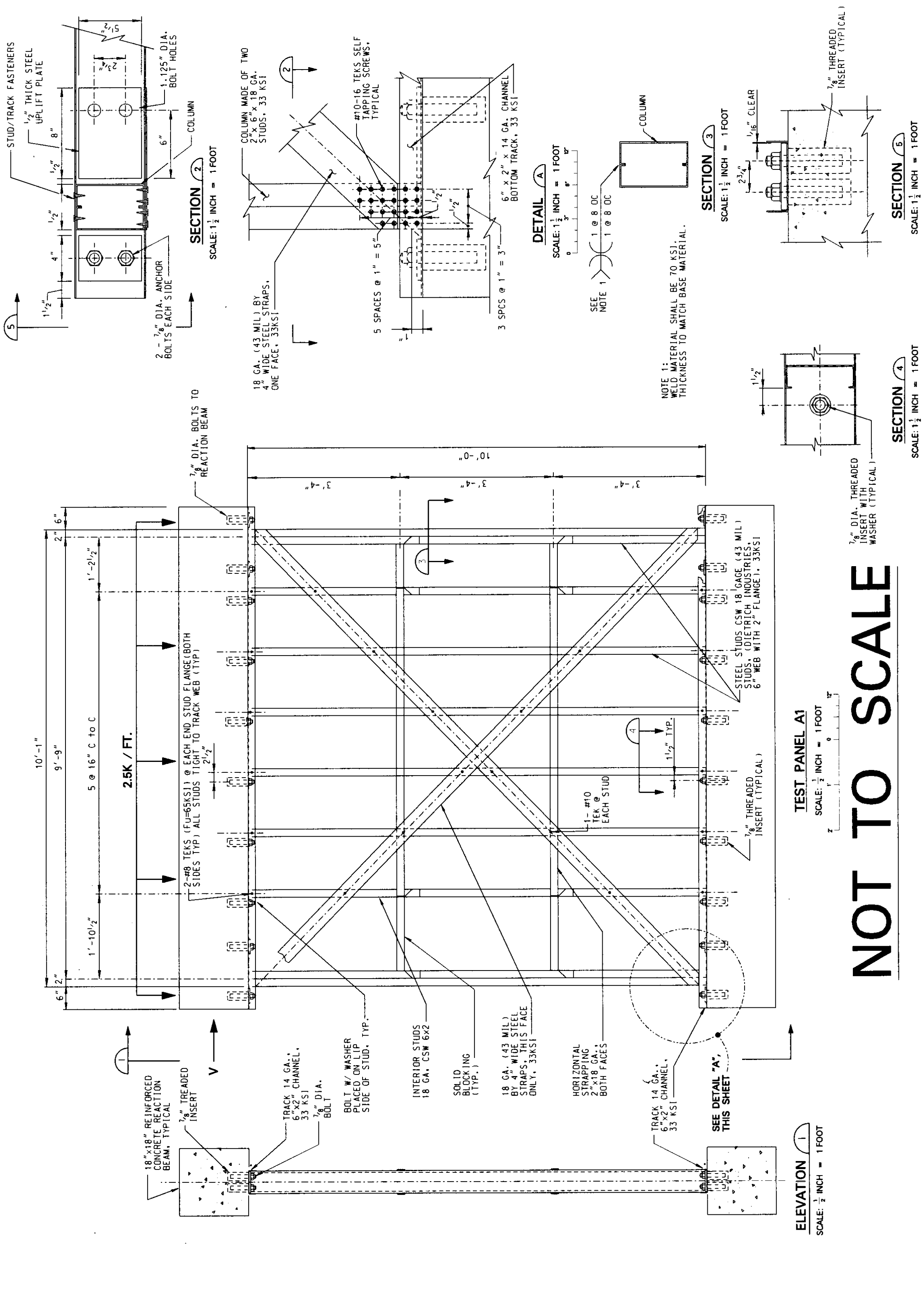

Figure B-2: Shear Wall Test Panel A1

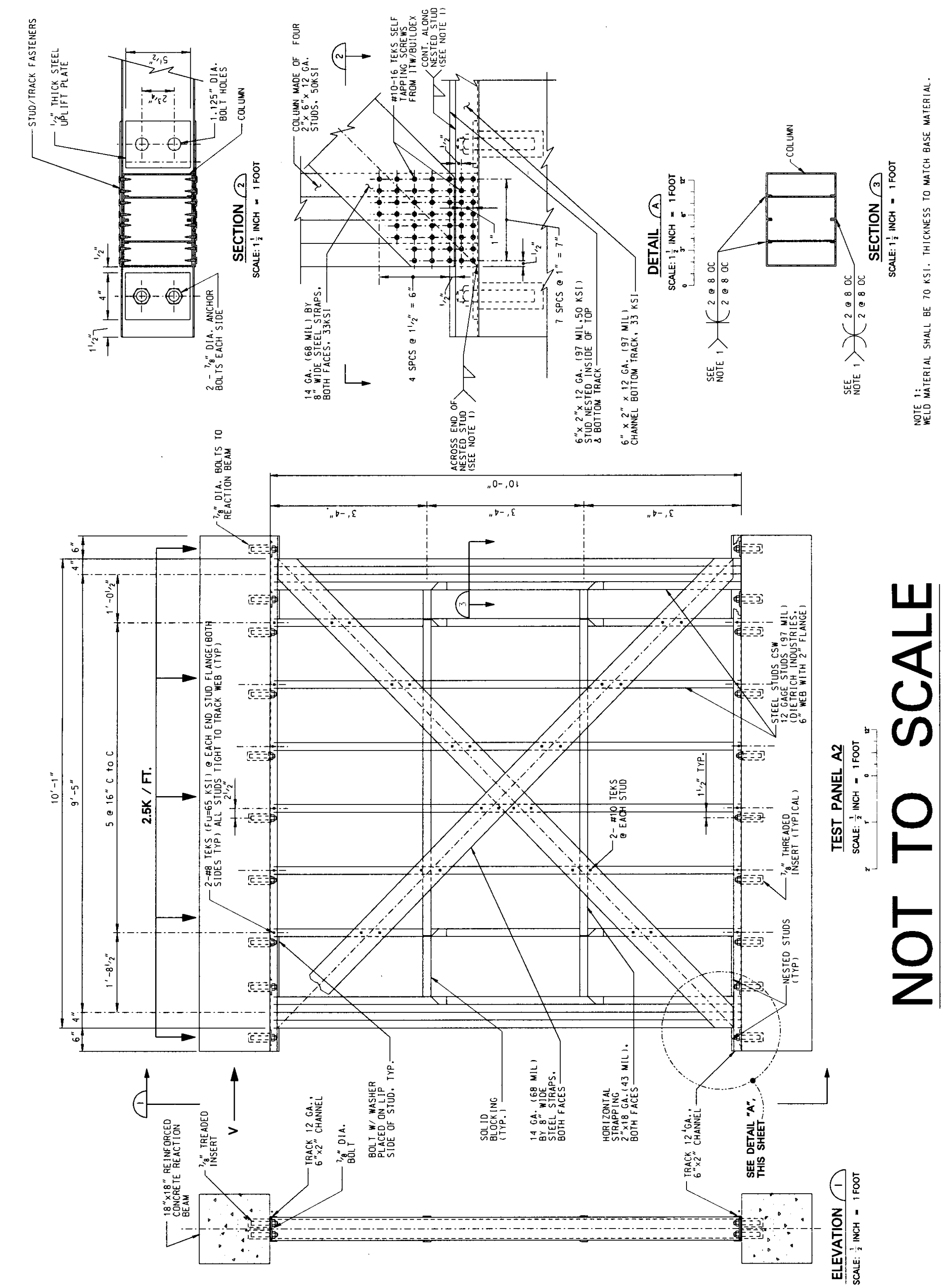

Figure B-3: Shear Wall Test Panel A2

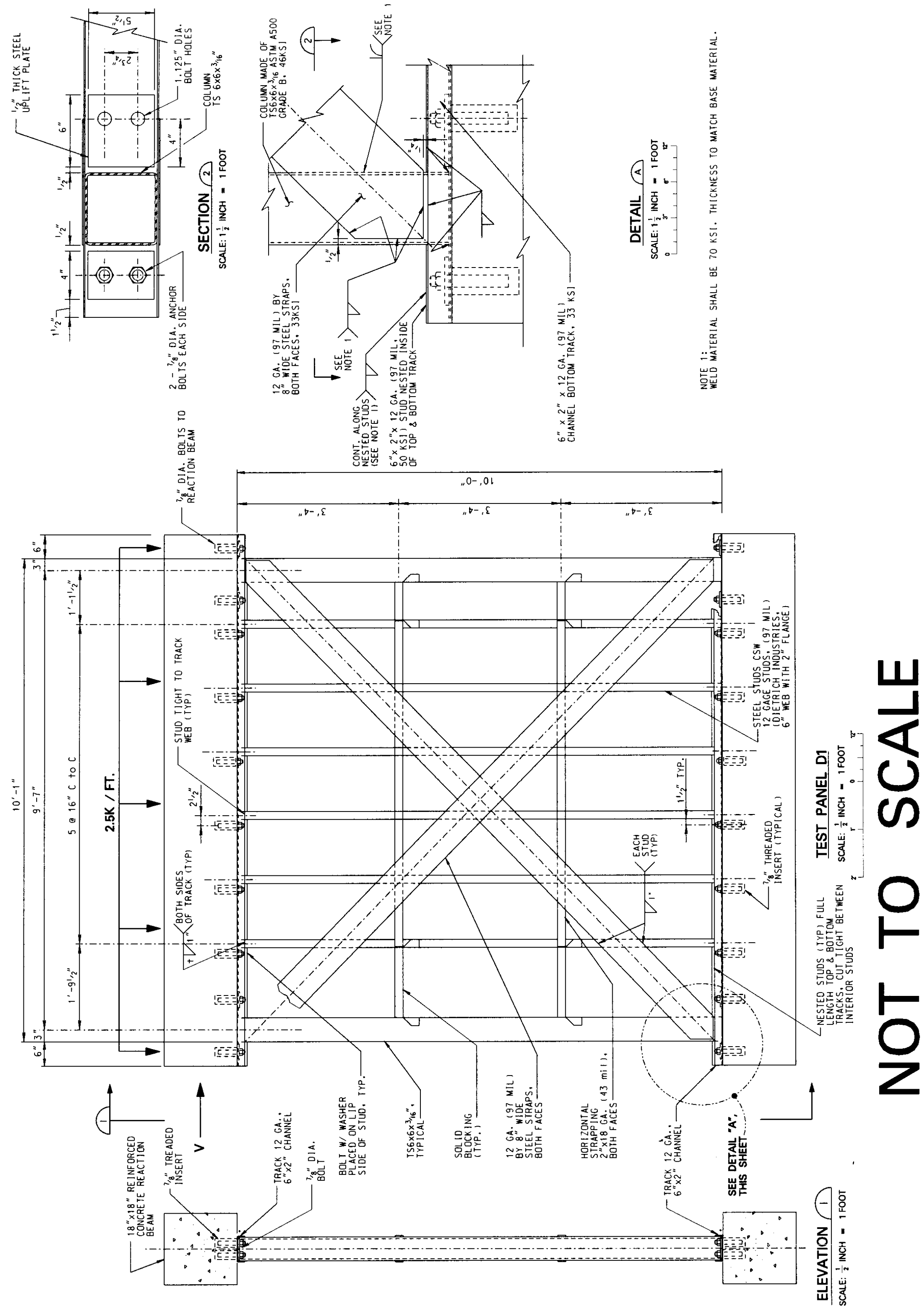

Figure B-4: Shear Wall Test Panel D1

APPENDIX C

FEMA 302 AND OTHER STANDARD GUIDANCE FOR COLD-FORMED STEEL SEISMIC DESIGN

C1. INTRODUCTION. Seismic use groups (FEMA 302, 1.3) are used to determine occupancy importance factors (FEMA 302, Table 1.4). Seismic use group III is for the most critical facilities as defined in FEMA 302, 1.3. This table is reproduced below:

Table C-1 Occupancy Importance Factors	
Seismic Use Group	I
I	1.0
II	1.25
III	1.5

TI 809-04 uses enhanced performance objectives to define seismic design forces for more critical facilities rather than the occupancy importance factors presented here.

C2. DEFINING GROUND MOTION. Seismic ground motions shall be defined according to FEMA 302 Chapter 4 and TI 809-04 Chapter 3. This paragraph defines ground motions for the maximum considered earthquake ground motions derived from Maps 1 through 24 (FEMA 302, Chapter 4). Spectral response acceleration at short periods, S_S and at 1 second, S_1 are obtained from Maps 1 through 24 of FEMA 302. For most regions of the nation, the maximum considered earthquake ground motion is defined with a uniform likelihood of exceedance of 2 percent in 50 years (2500 year return period)[1].

Site classifications shall be determined based on soil type (A through F), which may be based on shear wave velocity, ν_s average blow counts from standard penetration resistance test N [2] or unconfined shear strength, s_u of the soil (FEMA 302, 4.1.2). From the site classifications, values of site coefficients (F_a and F_v) are determined for the mapped spectral response acceleration values (FEMA 302, Table 4.1.1.4a and Table 4.1.1.4b). These tables are reproduced below:

Table C-2a Values of F_a as a Function of Site Class and Mapped Short-Period Maximum Considered Earthquake Spectral Acceleration[3]					
Site Class	Short Period Maximum Considered Response Spectral Acceleration				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	a[4]
F	a	a	a	a	a

[1] 1997 NEHRP, Part 2: Commentary (FEMA 303), Chapter 4 – Ground Motion, p. 37.
[2] Defined in ASTM D1536-84.
[3] Use straight line interpolation for intermediate values of S_S.
[4] a indicates that a site-specific geotechnical investigation and dynamic site response analyses shall be performed.

Table C-2b Values of F_v as a Function of Site Class and Mapped 1 Second Period Maximum Considered Earthquake Spectral Acceleration[5]					
Site Class	1 Second Period Maximum Considered Response Spectral Acceleration				
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \ge 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	a[6]
F	a	a	a	a	a

The maximum considered earthquake spectral response acceleration for short periods, S_{MS} and at 1 second, S_{M1} adjusted for site class effects are calculated as follows (FEMA 302, Eq. 4.1.2.4-1 and 4.1.2.4-2):

$$S_{MS} = F_a S_S \quad \text{(Eq C-1)}$$

and

$$S_{M1} = F_v S_1 \quad \text{(Eq C-2)}$$

These values define the elastic spectra. These values are reduced to define design earthquake spectral response acceleration at short periods, S_{DS} and at 1-second period, S_{D1} as follows (FEMA 302, Eq. 4.1.2.5-1 and 4.1.2.5-2):

$$S_{DS} = \frac{2}{3} S_{MS} \quad \text{(Eq C-3)}$$

and

$$S_{D1} = \frac{2}{3} S_{M1} \quad \text{(Eq C-4)}$$

From these terms a design response spectrum is developed as indicated in Figure C-1 (FEMA 302, Figure 4.1.2.6). For the natural period of the structure, T this spectrum defines values of effective acceleration. The three regions of this spectrum are defined as follows:

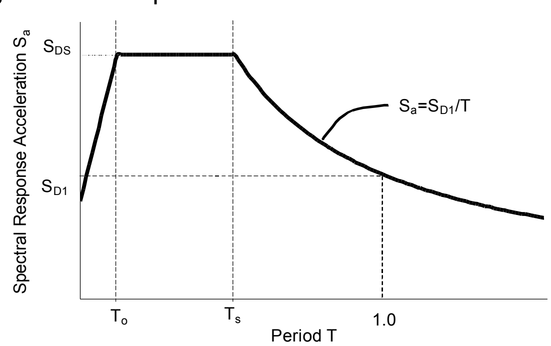

Figure C-1. Design response spectrum.

[5] Use straight-line interpolation for intermediate values of S_1.
[6] a indicates that a site-specific geotechnical investigation and dynamic site response analyses shall be performed.

For periods less than or equal to, T_0 the design spectral acceleration, S_a shall be (FEMA 302, Equation 4.1.2.6-1):

$$S_a = 0.6\frac{S_{DS}}{T_0}T + 0.4S_{DS} \quad \text{(Eq C-5)}$$

For periods greater than or equal to T_0 and less than or equal to T_S, the design spectral response acceleration , S_a, shall be taken as equal to S_{DS}.

For periods greater than T_S, the design spectral response acceleration, S_a, shall be (FEMA 302, Equation 4.1.2.6-3):

$$S_a = \frac{S_{D1}}{T} \quad \text{(Eq C-6)}$$

Where:

T = the fundamental period of the structure in seconds.
$T_0 = 0.2S_{D1}/S_{DS}$.
$T_S = S_{D1}/S_{DS}$.

C3. SEISMIC DESIGN CATEGORY. Each structure shall be assigned a seismic design category based on their Seismic Use Group and design response coefficients, S_{DS} and S_{D1} as indicated in the tables below (from FEMA 302 Tables 4.2.1a and 4.2.1b):

Table C-3a Seismic Design Category Based on Short Period Response Accelerations

Value of S_{DS}	Seismic Use Group		
	I	II	III
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

Table C-3b Seismic Design Category Based on 1 Second Period Response Accelerations

Value of S_{DS}	Seismic Use Group		
	I	II	III
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D
$0.75g \le S_1$	E	E	F

C4. STRUCTURAL CONFIGURATION AND REDUNDANCY. FEMA 302, Section 5.2.3 and 5.2.4 presents guidance on structural configurations and redundancy. Diaphragms are considered flexible if the maximum lateral deformation of the diaphragm exceeds twice the average story drift of the associated story (FEMA 302, 5.2.3.1).

A reliability factor, ρ, shall be defined for all structures based on the extent of structural redundancy in the lateral-force-resisting system. For structures in Seismic Design Categories A, B and C, the value for ρ shall be taken as 1.0. For structures in categories D, E and F, values for ρ shall be taken as the largest of the values of ρ_x calculated for each story of the structure "x" as follows (FEMA 302, Equation 5.2.4.2):

$$\rho_x = 2 - \frac{C1}{r_{max_x}\sqrt{A_x}} \qquad \text{(Eq C-7)}$$

Where:

r_{maxx} = the ratio of design story shear resisted by the single shear panel carrying the most shear force in the story to the total shear story, for a given direction of loading. Lateral loads shall be distributed to panels based on relative stiffness considering the interaction of panels with varying stiffness.

A_x = the floor area in m^2 (ft^2) of the diaphragm level immediately above the story.

ρ need not exceed 1.5, and may be used for any structure. The value of ρ shall not be taken as less than 1.0.

C1 = constant, 6.1 – metric, (20 – English)

C5. LOAD COMBINATIONS. Consideration of combinations of loads in the two orthogonal directions is not needed. The effects of gravity loads and seismic forces shall be combined in accordance with the factored load combinations as indicated below (ASCE 7[7]).

$$1.2D + 1.0E + 0.5L + 0.2S \qquad \text{(Eq C-8)}$$

and

$$0.9D + 1.0E \qquad \text{(Eq C-9)}$$

Where:

D = the dead load.

E = the effect of seismic load.

L = the live load – the load factor on L in Equation C-8 shall equal 1.0 for garages, areas occupied for public assembly, and all areas where the live load is greater than 4.79 kN/m^2 (100 psf).

S = the snow load – when the flat roof snow loads exceed 1.44 KN/m^2 (30 psf), the full design snow load shall be included in Equation C-8.

The effect of seismic loads, E shall be defined as follows, when the effect of gravity and seismic loads are additive (FEMA 302, 5.2.7):

$$E = \rho Q_E + 0.2 S_{DS} D \qquad \text{(Eq C-10)}$$

Where:

E = the effect of horizontal and vertical earthquake-induced forces.

ρQ_E = the maximum horizontal force that could be resisted by the bracing.

ρ = the system redundancy factor.

Q_E = the effect of horizontal seismic forces.

$0.2S_{DS}D$ = the vertical spectral acceleration effect of the seismic load.

S_{DS} = the design spectral response acceleration at short periods.

D = the effect of dead load.

The effect of seismic loads, E shall be defined as follows, when the effect of gravity and seismic loads counteract each other:

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[7] Minimum Design Loads for Buildings and Other Structures, ASCE 7-95, Section 2.3.2.

$$E = \rho Q_E - 0.2S_{DS}D \qquad \text{(Eq C-11)}$$

The effects of gravity load (dead, live and snow load) and seismic forces shall be combined as follows when the effect of gravity and seismic loads are additive, by combining Equations C-8 and C-10:

$$(1.2 + 0.2S_{DS})D + 0.5L + 0.2S + \rho Q_E \qquad \text{(Eq C-12)}$$

The effects of gravity load and seismic forces shall be combined as follows when the effect of gravity and seismic loads counteract each other, by combining Equations C-9 and C-11:

$$(0.9 - 0.2S_{DS})D + \rho Q_E \qquad \text{(Eq C-13)}$$

For both expressions in Equations C-12 and C-13, the total horizontal force is ρQ_E. This force alone defines the total lateral load that must be resisted by the shear panel diagonal straps or full panel sheets, and these elements should be sized based on this force.

The effect of seismic loads, E shall be defined as follows, to account for diagonal strap overstrength when the effect of gravity and seismic loads are additive:

$$E = \Omega_0 Q_E + 0.2S_{DS}D \qquad \text{(Eq C-14)}$$

Where:

Ω_0 = the system overstrength.

The effect of seismic loads, E shall be defined as follows, when the effect of gravity and seismic loads counteract each other:

$$E = \Omega_0 Q_E - 0.2S_{DS}D \qquad \text{(Eq C-15)}$$

The term $\Omega_0 Q_E$ calculated in Equations C-14 and C-15 (TI 809-04, Equations 4-6 and 4-7) need not exceed the maximum force that can be developed in the diagonal straps, based on the maximum estimated ultimate strength of these elements. This is expressed as follows:

$$\Omega_0 Q_E \le Q_u = F_{sumax} n_s b_s t_s \frac{W}{\sqrt{H^2 + W^2}} \qquad \text{(Eq C-16)}$$

Where:

F_{sumax} = the maximum ultimate stress of the diagonal straps, which equals 1.5 F_{su} for ASTM A653 Grade 33 steel (F_{su} = 310 MPa and 45 ksi) and 1.25 F_{su} for Grade 50 steel (F_{su} = 448 MPa and 65 ksi).
n_s = the number of diagonal straps.
b_s = the width of the diagonal straps.
t_s = the thickness of the diagonal straps.
W = the overall panel width.
H = the overall panel height (see Figure 3-2 for a schematic panel drawing showing W and H).

The effects of gravity load (dead, live and snow load) and seismic forces shall be combined as follows to account for diagonal strap overstrength, when the effect of gravity and seismic loads are additive, by combining Equations C-8 and C-14:

$$(1.2 + 0.2S_{DS})D + 0.5L + 0.2S + \Omega_0 Q_E \qquad \text{(Eq C-17)}$$

The effects of gravity load and seismic forces shall be combined as follows to account for diagonal strap overstrength, when the effect of gravity and seismic loads counteract each other, by combining Equations C-9 and C-15:

$$(0.9 - 0.2S_{DS})D + \Omega_0 Q_E \qquad \text{(Eq C-18)}$$

For both expressions in Equations C-17 and C-18, the total horizontal force is $\Omega_0 Q_E$. Every other term in these equations represent vertical loads. The shear panel systems should be analyzed based on the most critical load combination defined by either Equation C-17 or C-18. Each panel component (including all connections), other than the diagonal strap, should be designed based on these loads.

C6. EQUIVALENT LATERAL FORCE PROCEDURE. FEMA 302 and TI 809-04 present two methods for defining the structural response: the Equivalent Lateral Force Procedure (FEMA 302, 5.3) and the Modal Analysis Procedure (FEMA 302, 5.4). Paragraphs C7 through C9 presents the determination of base shear, period and vertical distribution of lateral forces using the Equivalent Lateral Force Procedure. Only this method is presented because of its simplicity and recognizing that typical cold-formed steel structures will likely be low rise construction so that first mode response will dominate the seismic response of the structures. However if deemed beneficial the modal analysis approach presented in FEMA 302 and TI 809-04 (Chapter 3-2.c.(2)) could be used.

C7. SEISMIC BASE SHEAR. Using the Equivalent Lateral Force Procedure, the seismic base shear, V in a given direction shall be determined according to the following equation (FEMA 302, 5.3.2):

$$V = C_s W \qquad \text{(Eq C-19)}$$

Where:
- C_s = the seismic response coefficient.
- W = the total dead load and applicable portions of other loads (see FEMA 302, 5.3.2).

The seismic response coefficient, C_s shall be determined according to the following equation:

$$C_s = \frac{S_{DS}}{R/I} \qquad \text{(Eq C-20)}$$

The value for C_s is calculated according to Equation C-20; and need not exceed the following:

$$C_s = \frac{S_{D1}}{T(R/I)} \qquad \text{(Eq C-21)}$$

but shall not be less than:

$$C_s = 0.1 S_{D1} I \qquad \text{(Eq C-22)}$$

nor shall it be taken as less than the following equation for Seismic Design Categories E and F:

$$C_s = \frac{0.5 S_1}{R/I} \qquad \text{(Eq C-23)}$$

C8. PERIOD DETERMINATION. The fundamental period of the building, T in the direction under consideration shall be defined using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis (FEMA 302, 5.3.3). Alternatively, T is permitted to be taken as the approximate fundamental period, T_a determined in accordance with the following requirements. The fundamental period, T shall not exceed the product of the coefficient for upper limit on calculated period, C_u from Table C-4 and the approximate fundamental period, T_a determined as follows:

$$T_a = C_T h_n^{3/4} \qquad \text{(Eq C-24)}$$

Where:

C_T = constant = 0.0731 – metric, (0.030 – English) for cold-formed steel shear panels with diagonal straps.

h_n = the height in meters (ft - English) above the base to the highest level in the structure.

Table C-4 Coefficient for Upper Limit on Calculated Period	
Design Spectral Response Acceleration at 1 Second, S_{D1}	Coefficient C_u
$S_{D1} < 0.1g$	1.7
$0.1g \le S_{D1} < 0.15g$	1.7
$0.15g \le S_{D1} < 0.2g$	1.5
$0.2g \le S_{D1} < 0.3g$	1.4
$0.3g \le S_{D1} < 0.4g$	1.3
$0.4g \le S_{D1}$	1.2

C9. VERTICAL DISTRIBUTION OF LATERAL SEISMIC FORCES. The vertical distribution of lateral seismic forces, F_x (kN or kip), induced at any level shall be determined from the following equations (FEMA 302, 5.3.4):

$$F_x = C_{vx} V \qquad \text{(Eq C-25)}$$

and

$$C_{vx} = \frac{W_x h_x}{\sum_{i=1}^{n} w_i h_i} \qquad \text{(Eq C-26)}$$

Where:

C_{vx} = the vertical distribution factor.

V = the total design lateral force or shear at the base of the structure (kN or kip).

w_i and w_x = the portion of the total gravity load of the structure, W located or assigned to level i or x.

h_i and h_x = the height (m or ft) from the base to level i or x.

The horizontal distribution of seismic story shear in any story, V_x (kN or kips) shall be determined from the following equation (FEMA 302, 5.3.5):

$$V_x = \sum_{i=x}^{n} F_i \qquad \text{(Eq C-27)}$$

Where:

F_i = the portion of the seismic base shear, V (kN or kips) induced at level i.

The seismic design story shear, V_x (kN or kips) shall be distributed to the various vertical elements of the seismic-force-resisting system in the story under consideration based on the relative lateral stiffness of the vertical resisting elements and the diaphragm.

C10. STRUCTURAL OVERTURNING RESISTANCE. The structure shall be designed to resist overturning effects caused by the seismic forces determined from Equation C-25. At any story, the increment of overturning moment in the story under consideration shall be distributed to the various vertical force resisting elements in the same proportion as the distribution of the horizontal shears to those elements (FEMA 302, 5.3.6).

The overturning moments at Level x, M_x (kN-m or kip-ft), shall be determined from the following equation:

$$Mx = \sum_{i=x}^{n} F_i (h_i = h_x) \qquad \text{(Eq C-28)}$$

Where:

F_i = the portion of the seismic base shear, V, induced at Level i.
h_i and h_x = the height (m or ft) from the base to Level i or x.

Foundations shall be designed for the foundation overturning design moment, M_f (kN-m or kip-ft) at the foundation-soil interface determined using Equation C-28 at the foundation level, multiplied by a reduction factor of 0.75.

C11. STORY DRIFTS AND P-DELTA EFFECTS. The story drifts and, member forces and moments due to P-delta effects shall be determined in accordance with the following guidance (FEMA 302, 5.37). Story drifts shall be calculated based on the application of design seismic forces to a mathematical model of the structure. The model shall include the stiffness and strength of all elements that are significant to the distribution of forces and deformations in the structure and shall represent the spatial distribution of the mass and stiffness of the structure. The design story drift, Δ shall be computed as the difference of the deflections at the center of mass at the top and bottom of the story under consideration. The deflections of level x, δ_x (mm or in) shall be determined according to the following equation:

$$\delta_x = \frac{C_d \delta_{xe}}{I} \qquad \text{(Eq C-29)}$$

Where:

δ_{xe} = the deflections determined by an elastic analysis (mm or in.) based on the forces defined in Equation C-25.

For determining compliance with the story drift limitations in Table 3-2, the deflections of Level x, δ_x (mm or in) shall be calculated as expressed in Equation C-29. For the purposes of this drift analysis only, the computed fundamental period, T in seconds, of the structure may be used without the upper bound limitations specified in Table C-4, when determining drift level seismic design forces.

The design story drift, Δ (mm or in) shall be increased by the incremental factor relating to the P-delta effects if required by the following guidance (FEMA 302, 5.3.7.2). The P-delta effects on story shears and moments, the resulting member forces and moments, and the story drifts induced by these effects are not required to be considered when the stability coefficient, θ as determined by the following equation is equal to or less than 0.10:

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d} \qquad \text{(Eq C-30)}$$

Where:

P_x = the total vertical design load at and above Level x (kN or kip). When calculating the vertical design load for purposes of determining P-delta, the individual load factors need not exceed 1.0.

Δ = the design story drift occurring simultaneously with V_x (mm or in).

V_x = the seismic shear force acting between Level x and x-1 (kN or kip).

The stability coefficient, θ shall not exceed θ_{max} determined as follows:

$$\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25 \qquad \text{(Eq C-31)}$$

Where:

β = the ratio of shear demand to shear capacity for the story between Level x and x - 1. This ratio may conservatively be taken as 1.0.

When the stability coefficient, θ is greater than 0.10 but less than or equal to θ_{max} the incremental factor related to P-delta effects, a_d shall be determined by rational analysis (see FEMA 303, Commentary, 5.3.7). To obtain the story drift for including the P-delta effects, the design story drift, Δ shall be multiplied by 1.0/(1 - θ). When θ is greater than θ_{max} the structure is potentially unstable and shall be redesigned.

C12. DIAGONAL STRAP DESIGN. From the values of seismic story shear, V_x and additional shear force due to torsion, the shear panel dimensions are defined and diagonal straps designed. The straps are tension only members and their design strength is defined by the following equation (AISI, C2, p. V 45):

$$\phi_t A_n F_{sy} = \phi_t \sum (b_s t_s) F_{sy} \qquad \text{(Eq C-32)}$$

Where:

ϕ_t = the resistance factor for tensile members (0.95).

A_n = the cross-sectional area of the all diagonal straps in tension ($b_s t_s$).

b_s= the width of an individual diagonal strap.

t_s = the thickness of an individual strap.

F_{sy} = the design yield strength of the strap.

The shear panel lateral yield capacity, Q_{sy} when the diagonal straps are the sole lateral-load-resisting element is calculated as follows:

$$Q_{sy} = n_s b_s t_s F_{sy} \left(\frac{W}{\sqrt{H^2 + W^2}} \right) \qquad \text{(Eq C-33)}$$

Where:

W = the width of a trial shear panel.

H = the height of a trial shear panel.

The shear panel design strength, $\phi_t Q_{sy}$ must be greater than the seismic story shear, V_x and additional shear force due to torsion, Qsi for all shear panels resisting in the frame of the building for which these forces are applied. This is expressed as:

$$\phi_t Q_{sy} = \phi_t \sum\left[n_s b_s t_s F_{sy}\left(\frac{W}{\sqrt{H^2 + W^2}}\right)\right] \geq V_x + Q_{si} \qquad \text{(Eq C-34)}$$

The number of shear panels, panel width, height, and strap size and strength shall be designed according to Equation C-34 to meet minimum lateral yield capacity. All diagonal strap material must be ASTM A653 steel. Diagonal straps may not use re-rolled steel, because the re-rolling strain hardens the material, increasing material strength variability and reducing elongation (see USACERL TR FL-XX, Chapter 4 for a discussion of this concern).

C13. COLUMN AXIAL CAPACITY. The column axial design strength, P shall be determined as follows for columns built-up with cold-formed steel studs or individual structural tubing members (AISI, C4, Concentrically Loaded Compression Members):

$$P = \phi_c A_e F_n \qquad \text{(Eq C-35)}$$

Where:

ϕ_c = the resistance factor for compression, which equals 0.85.
A_e = the effective area at the stress F_n.
F_n = the nominal strength of the column, determined as follows:

For $\lambda_c \leq 1.5$

$$F_n = (0.658^{\lambda_c^2})F_{cy} \qquad \text{(Eq C-36)}$$

For $\lambda_c > 1.5$

$$F_n = \left[\frac{0.877}{\lambda_c^2}\right]F_{cy} \qquad \text{(Eq C-37)}$$

Where:

$$\lambda_c = \sqrt{\frac{F_{cy}}{F_e}} \qquad \text{(Eq C-38)}$$

Where:

F_{cy} = the column design yield strength.

$$F_e = \frac{\pi^2 E}{\left(KL/r\right)^2} \qquad \text{(Eq C-39)}$$

Where:

E = the modulus of elasticity.
K = the effective length factor.
L = the unbraced length of the column.
r = the radius of gyration of the full, unreduced column cross section, calculated as follows:

$$r = \sqrt{I/A} \qquad \text{(Eq C-40)}$$

The effective area, A_e is calculated as follows for columns built-up from cold-formed steel studs such that they form a closed section or structural tube columns (AISI, C4):

$$A_e = A_c - nt_c(w - b) \qquad \text{(Eq C-41)}$$

Where:

A_c = the nominal column area.
n = the number of studs making up the column, or is 2 when using structural tube columns.
t_c= the thickness of the stud material used in the built-up columns, or the thickness of the structural tube column.
w = the flat width of the stud web making up the built-up columns, or the width of the structural tube face perpendicular to the plane of the panel. Assuming the outside radius of the stud corners is twice the thickness, t_c this may be calculated as follows:

$$w = d_s - 4t_c \qquad \text{(Eq C-42)}$$

d_s = the depth of the studs making up the built-up columns, or the structural tube width perpendicular to the plane of the panel,
b = the effective width and shall be determined as follows (AISI, B2.2):

$$\text{for} \quad 0.5 \geq \frac{d_h}{w} \geq 0, \text{and} \frac{w}{t_c} \leq 70 \quad \text{and}$$

the distance between centers of holes ≥ 0.5w and ≥ $3d_h$,

$$b = w - d_h \quad \text{when} \quad \lambda \leq 0.673 \qquad \text{(Eq C-43)}$$

$$b = \frac{w\left[1 - \frac{0.22}{\lambda} - \frac{0.8d_h}{w}\right]}{\lambda} \quad \text{when } \lambda > 0.673 \qquad \text{(Eq C-44)}$$

Where:

d_h = the diameter of holes.
λ = a slenderness factor defined as follows (AISI, B2.1):

$$\lambda = \frac{1.052}{\sqrt{k}}\left(\frac{w}{t_c}\right)\sqrt{\frac{F_n}{E}} \qquad \text{(Eq C-45)}$$

Where:

k = the plate buckling coefficient, equal to 4 for the studs making up the built-up columns or structural tube columns.

C14. COLUMN SHEAR CAPACITY. The column shear design strength, V_c shall be determined as follows for columns built-up with cold-formed steel studs or individual structural tubing members (AISI, C3.2, Strength for Shear Only):

$$\text{For} \quad \frac{h}{t_c} \le 0.96\sqrt{\frac{Ek_v}{F_{cy}}}$$

$$V_C = \phi_v 0.60 F_{cy} h_c n_s t_c \qquad \text{(Eq C-46)}$$

Where:

- h = the depth of the flat portion of the web, which equals the stud flange width for the built-up columns.
- k_v = the shear buckling coefficient, which equals 5.34 for both built-up and structural tubing columns.
- ϕ_v = 1.0
- h_c = the depth of the column, which is the column width in the in-plane direction of the panel.
- n_s = the number of faces of a shear panel with diagonal straps (i.e., 1 or 2).

C15. CONNECTION SHEAR AND PULL-OVER. The design shear (AISI E4.3.1) and pull-over per screw (AISI E4.4.2), P_s shall be calculated as follows:

$$P_s = \phi_s \min(P_{ns} \text{and} P_{nov}) \qquad \text{(Eq C-47)}$$

Where, the nominal shear strength per screw, P_{ns}, shall be determined as follows:

For $t_2/t_1 \le 1.0$, P_{ns} shall be taken as the smallest of:

$$P_{ns} = 4.2\sqrt{t_2^3 d}F_{u2} \quad \text{tilting mode of failure} \qquad \text{(Eq C-48)}$$

$$P_{ns} = 2.7 t_1 d F_{u1} \quad \text{bearing mode of failure} \qquad \text{(Eq C-49)}$$

$$P_{ns} = 2.7 t_2 d F_{u2} \quad \text{bearing mode of failure} \qquad \text{(Eq C-50)}$$

For $t_2/t_1 \ge 2.5$, P_{ns} shall be taken as the smaller of:

$$P_{ns} = 2.7 t_1 d F_{u1} \quad \text{bearing mode of failure} \qquad \text{(Eq C-51)}$$

$$P_{ns} = 2.7 t_2 d F_{u2} \quad \text{bearing mode of failure} \qquad \text{(Eq C-52)}$$

For $1.0 < t_2/t_1 < 2.5$, P_{ns} shall be determined by linear interpolation between the two cases above.

Where:

- ϕ_s = the screw resistance factor for shear, equal to 0.5.
- d = the nominal screw diameter.
- t_1 = the thickness of the member in contact with the screw head.
- t_2 = the thickness of the member not in contact with the screw head.
- F_{u1} = the ultimate tensile strength of the member in contact with the screw head.
- F_{u2} = the ultimate tensile strength of the member not in contact with the screw head.

The nominal shear strength per screw, P_{ns} may also be determined by AISI Tables IV-7a and IV-7b for connections to various sheet thicknesses for sheets with ultimate strengths of 310 MPa (45 ksi) and 448 MPa (65 ksi) (ASTM A653, Grade 33 and 50 respectively). These tables may only be used if ultimate strengths of the materials being connected are the same.

The nominal pull-over strength, P_{nov} shall be calculated as follows:

$$P_{nov} = 1.5 t_1 d_w F_{u1} \qquad \text{(Eq C-53)}$$

Where:

d_w = the larger of the screw head diameter or the washer diameter, and shall not be taken as larger than 13 mm (½ inch).

C16. WELDED CONNECTION DESIGN. Diagonal strap-to-column connection fillet weld design based on AISI (E2.4 Fillet Welds) is summarized below. The design shear strength for loading in the longitudinal direction, P_L shall be determined as follows:

For $L/t < 25$
$$P_L = \left(1 - \frac{0.01L}{t}\right)\phi t L F_u \qquad \text{(Eq C-54)}$$

Where:

ϕ = 0.60.
L = the length of fillet weld.
t = the least value of the thicknesses of the two members being welded.

For $L/t \geq 25$
$$P_L = 0.75\phi t L F_u \qquad \text{(Eq C-55)}$$

Where:

ϕ = 0.55

The design shear strength of loading in the transverse direction, P_T shall be determined as follows:

$$P_T = \phi t L F_u \qquad \text{(Eq C-56)}$$

Where:

ϕ = 0.60.

For fillet welds to heavy strap material, thicker than 0.150 inches, the design shear strength for both longitudinal and transverse loading due to weld failure, P_W[8] shall not exceed the following:

$$P_W = 0.75\phi t_w L F_{xx} \qquad \text{(Eq C-57)}$$

Where:

ϕ = 0.60.
t_w = the effective throat, equal to 0.707 times the least leg of the weld in-plane or out-of-plane of the materials being welded. A larger effective throat shall be permitted if measurement shows that the welding procedure to be used consistently yields a larger value of t_w.
F_{xx} = the weld metal strength designation in AWS electrode classification.

C17. ANCHOR FLARE BEVEL GROVE WELD DESIGN. Column-to-anchor flare bevel grove weld design based on AISI (E2.5 Flare Grove Welds) is summarized below. The design strength of the longitudinal loaded flare bevel grove weld, P_G shall be determined as follows:

[8] AISI Commentary, E2.4, indicates that this equation is needed to cover the possibility of weld failure through the throat of the weld material, because tests that ensured tearing in the plate were not conducted on plates thicker than 0.15 inches.

For $t_c \le t_w < 2t_c$ (single shear) $$P_G = 0.75\phi_G t_c L F_{cu}$$ (Eq C-58)

For $t_w \ge 2t_c$ (double shear) $$P_G = 1.5\phi_G t_c L F_{cu}$$ (Eq C-59)

Where:

ϕ_G = the resistance factor for flare grove welds, equal to 0.55.
t_c = the thickness of the column material.
L = the length of the flare bevel grove weld.
F_{cu} = the ultimate strength of the column steel.

C18. ANCHOR BOLT CONE FAILURE. Embedded anchors shall be used for all anchor bolts described here. The anchor bolt cone failure design strength, P_c shall exceed the applied tensile force per bolt, P_{tAB} (Equation 3-37). Either one or two anchor bolts may be installed on both sides of the columns (i.e., n_{AB} equal to 2 or 4). If only one bolt is installed, the cone failure surface will be that of a simple cone. If two bolts are installed, the critical failure surface will be the minimum failure surface defined by two independent cones or a surface that accounts for the overlap of two cones. If the two bolts on the same side of the column are close enough relative to the bolt embedment length, then the combined surface will control. The column anchor bolts at the outside of the shear panel will always be more highly stressed than those at the inside. These bolts will be more critically loaded by uplift forces due to the direction of diagonal strap forces and moment in the column. A cone failure surface including bolts on both sides of the columns will never be more critical than the cone for bolts only at the outside of the shear panel. Therefore, only the cone with bolts on the outside are considered. The anchor bolt cone failure design strength, P_c (in pounds) is determined by[9]:

$$P_c = 4\phi_c \sqrt{f'_c}\, A_c \qquad \text{(Eq C-60)}$$

Where:

ϕ_c = the cone strength reduction factor - a value of 0.85 for uncracked concrete.
f'_c = the specified concrete compressive strength in psi
A_c = the minimum of the area of a single anchor bolt stress cone (A_{c1}) or the summation of the combined failure surface for two overlapping stress cones divided by two (A_{c2}), in inches^2. The area of individual stress cones is dependent on the angle of cone failure. In the case of expansion anchors this angle varies from about 60 degrees (measure from the axis of the anchor) for short embedments ($l_{AB} \le 2$ inches) to 45 degrees for $l_{AB} \ge 6$ inches. In this guidance this angle will conservatively be set equal to 45 degrees. Then the radius of the cone at the concrete surface, r_c is equal to anchor embedment length, l_{AB}. The area of an individual stress cone failure surface, A_{c1} shall be calculated as follows:

$$A_{c1} = \pi r_c \sqrt{r_c^2 + l_{AB}^2} = \sqrt{2}\pi l_{AB}^2 \qquad \text{(Eq C-61)}$$

The area of the combined failure surface for two overlapping stress cones, divided by two for an individual anchor bolt, A_{c2} shall be calculated as follows:

$$A_{c2} = \frac{\pi r_c + 2d_{cc}}{2}\sqrt{r_c^2 + l_{AB}^2} = \frac{\sqrt{2}}{2} l_{AB}\left(\pi l_{AB} + 2d_{cc}\right) \qquad \text{(Eq C-62)}$$

[9] ACI 355.1R-91, Equation 3.2.

The anchor bolts shall not be installed too close to the edge of a concrete beam or slab, or edge failure could occur before developing the cone strength. The minimum distance from the center of an anchor bolt to the edge of the concrete to prevent side cone failure, m (in inches) is determined as follows:[10]

$$m = d_{AB}\sqrt{\frac{F_t}{73\sqrt{f'_c}}} \qquad \text{(Eq C-63)}$$

[10] ACI 355.1R-91, Equation 3.3.

APPENDIX D

SEISMIC DESIGN EXAMPLE
(English I-P units only)

D1. EXAMPLE DESIGN PROBLEM. An example problem is presented here to demonstrate the design process presented in Chapter 3 and Appendix C. Shear panels will be designed in the short direction of the building only to illustrate the design process. In an actual building the lateral load resisting system must be designed in both directions. This example is a barracks-type building that will be designed for construction at Fort Lewis, located between Tacoma and Olympia, Washington. This building is similar to a Prototype 3 Story Steel Stud Framed Barracks Building for Seismic Zones 0 – 2[1]. The reader will be referred to tabular data and equations presented in Chapter 3 and Appendix C. When needed, FEMA 302 guidance will be referenced.

The barracks building has a Seismic Use Group of I (FEMA 302, 1.3), which gives it an Occupancy Importance Factor, I, of 1.0 (see Table C-1).

D2. GROUND MOTION DEFINITION. The maximum considered earthquake ground motions are determined from spectral response acceleration Maps 9 and 10 (for the Pacific Northwest). The spectral response acceleration for short periods, S_S, is 1.2 g (Map 9). The spectral response acceleration for 1 second, S_1, is 0.39 g (Map 10). Table D-1 summarizes these values. These values are determined by interpolating between the map contours for the Fort Lewis location. The soil conditions are unknown, so a reasonable worst-case site classification of D is used. Values of Site Coefficients, F_a and F_v, are calculated based on straight-line interpolation from the values in Tables C-2a and C-2b, and are shown in Table D-1. Values for the maximum considered earthquake spectral response acceleration for short periods, S_{MS} and at 1 second, S_{M1} adjusted for site class effects, are calculated using Equations C-1 and C-2, and are shown in Table D-1. Design earthquake spectral response acceleration at short periods, S_{DS} and 1 second period, S_{D1} are calculated using Equations C-3 and C-4, and are shown in Table D-1.

Table D-1. Earthquake Ground Motion Definition Summary for Fort Lewis.

Importance Factor, I	1.0
Short Period Spectral Response Acceleration, S_S	1.2 g
1 Second Spectral Response Acceleration, S_1	0.39 g
Site Classification	D
Site Coefficient, F_a	1.02
Site Coefficient, F_v	1.62
Adjusted Short Period Spectral Response Acceleration, S_{MS}	1.22 g
Adjusted 1 Second Spectral Response Acceleration, S_{M1}	0.63 g
Design Short Period Spectral Response Acceleration, S_{DS}	0.82 g
Design Short Period Spectral Response Acceleration, S_{D1}	0.42 g
T_0	0.103 seconds
T_S	0.516 seconds
Assumed Design Spectral Response Acceleration, S_a	0.82 g
Seismic Design Category	D
Response Modification Factor, R	4
Deflection Amplification Factor, C_d	3.5

A design response spectrum is developed from these terms, as described in Appendix C, Paragraph C2, using Equations C-5 and C-6, and plotted in Figure D-1. For the natural period of the structure, T, this spectrum defines values of effective acceleration. The natural period of the barracks building, T, will almost certainly fall between T_0 and T_S, defined in Appendix C, Paragraph C2, so that the

[1] U.S. Army Corps of Engineers Barracks Prototype Department of the Army, for the National Association of Architectural Metal Manufactures (NAAMM), by Matsen Ford Design, Drawings Dated 1/3/97.

design spectral acceleration S_a will equal S_{DS}. Values for T_0 and T_S are shown in Table D-1. After the building frame is designed, the building natural period will be calculated to ensure that it falls between T_0 and T_S, and corrections will be made if needed.

D3. SEISMIC DESIGN CATEGORY. The seismic design category for the barracks building is determined from Tables C-3a or C-3b, based on the seismic use groups and values of S_{DS} and S_{D1}. If the tables give different categories, the larger letter is chosen. For the barracks building, the seismic design category is D (see Table D-1).

D4. STRUCTURAL DESIGN CRITERIA. The lateral-load-resisting system of the barracks building will be designed with cold-formed steel shear panels with diagonal straps acting as the sole lateral-load-resisting element. Values of the response modification factor, R and deflection amplification factor, C_d are taken from Table 3-1 and shown again in Table D-1.

The diaphragms of the barracks buildings are reinforced concrete and are considered rigid. The reliability factor, ρ_x, is calculated using Equation C-7, which for the barracks building for every floor level gives:

$$\rho_x = 2 - \frac{20}{r_{max\,x}\sqrt{A_x}} = 2 - \frac{20}{\frac{1}{18}\sqrt{8971\,\text{sq.ft.}}} = -1.8 \qquad \text{(Eq D-1)}$$

The value of ρ_x shall not be taken as less than 1.0. Therefore no correction is needed for lateral-load-resisting system reliability.

D5. BARRACKS BUILDING LOAD CALCULATIONS. The effects of gravity load (dead, live, and snow) and seismic forces shall be combined as defined by Equations C-12 and C-13. As explained in Appendix C, Paragraph C5, the total lateral force that must be resisted by the shear panel diagonal straps is simply defined by ρQ_E in these equations. In the case of the barracks building this becomes Q_E, and the diagonal straps are first sized based on this force.

The barracks building will be designed to act independently in the two orthogonal directions. Figures D-2 and D-3 show schematic drawings of the barracks building. Figure D-2 shows the plan view and long-direction elevation. Figure D-3 shows the short-direction elevation of the building. Table D-2 summarizes the weight calculations for the entire building using spreadsheet calculations. These weights include roof and floor dead load (20 and 40 psf, respectively); exterior wall weight (10 psf); interior wall weight (10 psf); brick veneer weight (40 psf); and room and corridor live load (40 and 80 psf, respectively).[2] The brick veneer is self-supporting for gravity loads, and vertical and in-plane lateral seismic forces. The building lateral-load-resisting system (shear panels) does resist out-of-plane lateral seismic forces from the brick veneer weight. Therefore, the out-of-plane long-direction brick veneer lateral seismic forces are resisted by the short-direction shear panels.

The short-direction shear panels will be placed at every bay (20 feet, 6-5/8 inches spacing) of the building as shown in Figure D-2, for a total of nine short-direction frames. A trial shear panel configuration will be assumed in which two shear panels are placed at every frame, as shown in Figure D-3. Figure D-3 shows that two shear panels will be placed against the perpendicular outside walls of the building. Shear panels will be located in the same bay at each floor level, with decreasing capacity at the higher floor levels.

[2] Barracks Prototype Drawings, Sheet C-1.

Table D-2. Barracks Building Weight Calculations.

Panel Level	Floor Dead Load (psf)	Floor Length (ft)	Floor Width (ft)	Floor Area (ft^2)	Total Floor Dead Load, D (kips)	Story Height (ft)	Exterior Walls (psf)	Total Exterior Walls, EW (kips)	Interior Walls (psf)	Total Interior Walls, IW (kips)	Total Dead Load D_T=D+EW+IW (kips)	Self Supporting for gravity Brick Veneer (psf)	Long Direction Brick Veneer B_L (kips)	Short Direction Brick Veneer B_S (kips)	Room Live Load (psf)	Room Area (ft^2)	Corridor Live Load (psf)	Corridor Area (ft^2)	Total Floor Live Load, L (kips)
Roof																			
3rd	20	164.42	54.67	8988	179.762	4.2	10	18.5	10	30.09	228.4	40	55.6	18.5	0	7892	0	1096	0
2nd	45	164.42	54.67	8988	404.465	9.0	10	39.3	10	63.74	507.5	40	117.8	39.2	40	7892	80	1096	403
1st	45	164.42	54.67	8988	404.465	9.3	10	40.7	10	66.11	511.3	40	122.2	40.6	40	7892	80	1096	403

The ground snow load, p_g, for Fort Lewis is 20 psf[3]. The flat-roof snow load, p_f, is calculated as follows (ASCE 7-95, Eq 7-1)[4]:

$$p_f = 0.7C_eC_tIp_g = 0.7(0.9)(1.0)(1.0)(20psf) = 12.6psf \qquad \text{(Eq D-2)}$$

Where:

C_e = the exposure factor (ASCE 7-95, Table 7-2), which for an exposure category C, fully exposed roof is 0.9.
C_t = the thermal factor (ASCE 7-95, Table 7-3), which is taken as 1.0.
I = the importance factor (ASCE 7-95, Table 7-4), which for Category II of the barracks building is 1.0.

However, the flat-roof snow load shall not be less than the ground snow load multiplied by the importance factor (p_gI), so that the p_f = 20 psf. The sloped-roof snow load, p_s is calculated as follows (ASCE 7-95, Eq 7-2):

$$p_s = C_sp_f = (0.75)(20psf) = 15psf \qquad \text{(Eq D-3)}$$

Where:

C_s = the roof slope factor (ASCE 7-95, Figure 7.2), which is 0.75 for the barracks building with a 5/12 roof slope.

The snow load will not be used in this example because the flat roof snow load does not exceed 30 psf, and therefore is not included in load combinations that include seismic forces.

D6. EARTHQUAKE FORCE DEFINITION. Seismic forces are now defined based on the equivalent lateral force procedure (see Appendix C, Paragraphs C6 through C9). The seismic base shear, V in the direction of the shear walls is given by (Equation C-19):

$$V = C_sW \qquad \text{(Eq D-4)}$$

The seismic response coefficient, C_s (Equation C-20) is calculated with the variables given in Table D-1, which becomes:

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.82g}{4/1.0} = 0.204g \qquad \text{(Eq D-5)}$$

The value of for C_s need not exceed the following (Equation C-21), where T = T_a (see Equation D-8):

[3] ASCE 7-95, Chapter 7 and Chapter 7 Commentary.
[4] NEHRP, Section 5.3.2, states that in areas where the design flat roof snow load does not exceed 30 psf, the effective snow load is permitted to be taken as zero. The Commentary to the 1997 NEHRP (FEMA 303) Section 5.32, states that "snow loads up to 30 psf are not considered," in the weight, W, used to calculate the lateral earthquake loads.

$$C_s = \frac{S_{D1}}{T\left(R/I\right)} = \frac{0.42g}{0.36(4/1.0)} = 0.292g \quad \text{(Eq D-6)}$$

but shall not be less than (Equation C-22):

$$C_s = 0.1S_{D1}I = 0.1(0.42g)(1.0) = 0.042g \quad \text{(Eq D-7)}$$

The approximate fundamental period, T_a, in seconds is calculated using the following equation (Equation C-24):

$$T_a = C_T h_n^{3/4} = (0.030)(27)^{3/4} = 0.36 \text{ seconds} \quad \text{(Eq D-8)}$$

Where:

C_T = 0.030 for cold-formed steel shear panels with diagonal straps.
h_n = the height, which is 27 feet to the top of the shear walls for the barracks building.

This approximate period, T_a is used for the fundamental period, T in Equation D-6 without correction.

D7. SHORT-DIRECTION EARTHQUAKE FORCE DEFINITION. The total dead load and applicable portions of other loads, W are calculated from the loads presented in Table D-2 as follows:

$$W = D_T + B + 0.25L = D + EW + IW + B + 0.25L \quad \text{(Eq D-9)}$$

For the short direction of the building this weight, W_S becomes:

$$W_S = D_T + B_L + 0.25L = D + EW + IW + B_L + 0.25L \quad \text{(Eq D-10)}$$

Where:

D_T = the total dead load.
B = the brick veneer weight.
L = the live load.
D = the floor and roof dead load.
EW = the exterior wall weight.
IW = the interior wall weight.
B_L = the brick veneer weight in the long direction of the building carried by the shear panels in the short direction of the building.

The cumulative total weight in the short direction of the building, W_S at the first floor is equal to 1744 kips, as shown in Table D-3.

The base shear in the short direction of the building, V_S, is now calculated from Equation C-19:

$$V_S = C_s W_S = (0.204g)(1744\text{kips}) = 356\text{kips} \quad \text{(Eq D-11)}$$

Table D-3. Short-Direction Lateral Seismic Force Calculations for the Barracks Building.

Panel Level	Short Direction Total Weight W_S (k-mass)	Seismic Response Coefficient C_s (g)	Short Dir Base Shear V_S (kips)	Height at Floor Level h_{xS} or h_{xL} (ft)	Short Dir Vertical Distribution Factor C_{vxS}	Number Frames in Short Dir n_S	Short Dir Lateral Seismic Force/frame F_{xS} (kips)	Accidental Torsion M_{tax} (kip-ft)	Max. Add Shear due to Acc Torsion Q_{sic} (kips)	Short Dir Seismic Story Shear V_{xS} (kips)
Roof										
3rd	284			27.042	0.276	9	10.895	1040	2.529	13.424
Cumulative	284									
2nd	726			18.583	0.484	9	19.142	1837	4.469	37.035
Cumulative	1010									
1st	734			9.125	0.240	9	9.506	912	2.218	48.758
Cumulative	1744	0.204	356							

The vertical distribution of lateral seismic forces in the short direction, F_{xS}, induced at any level shall be determined using Equation C-25. These values are determined based on the vertical distribution factor in the short direction, C_{vxS}, calculated in Equation C-26. Values for W_{xS}, h_x, w_i, and h_i used in Equation C-26 are given in Table D-3. The short-direction lateral seismic forces, F_{xS}, shown in Table D-3 are the lateral force per frame in the short direction. There are nine frames in the short direction, n_S[5], so that lateral force per frame is calculated as follows:

$$F_{xS} = \frac{C_{vxS} V_S}{n_S} \qquad \text{(Eq D-12)}$$

The barracks building is very regular in plan, so the center of rigidity, C_R in both directions should be at the center of the building. The accidental torsion is accounted for by offsetting the center of mass, C_M, 5 percent in both directions in plan at each floor level (see Figure D-2). The total mass at each floor level in each direction (long and short) is multiplied by the 5 percent of the building dimension in that direction to calculate the accidental torsional moment, M_{ta} at each floor level. Similar to the lateral seismic forces, the accidental torsional moments, M_{tax} are distributed along the floors of the building according to the vertical distribution factor given in Equation C-26, which is expressed as follows:

$$M_{tax} = 0.5[V_S C_{vxS}(\text{FloorLength}) + V_L C_{vxL}(\text{FloorWidth})] \qquad \text{(Eq D-13)}$$

Where:

C_{vxL} = vertical distribution factor in the long direction.
V_L = the base shear in the long direction.

Table D-3 gives values for accidental torsional moments, M_{tax} at each floor level.

The torsional resistance, M_{tr} (see Equation 3-3) is proportional to the square of the distance from the center of resistance to the plane of each panel. The torsional resistance is also proportional to the lateral stiffness of each panel. Therefore, because the barracks building is very long in one direction, the shear panels in the short direction near the ends of the building will dominate the torsional resistance. For this example it will be assumed that all torsional resistance comes from the shear panels in the short direction. The torsional resistance from all shear panels, M_{tr}, in the short direction can be expressed as follows (from Equation 3-3):

[5] The symbol for the number of frames in the short direction, n_S, must not be confused with the number of faces with diagonal straps on a given shear panel, n_s.

$$M_{tr} = \sum_{i=1}^{n} r_i^2 k_{si} q = 4\left[(20.5')^2 + (2x20.5')^2 + (3x20.5')^2 + (4x20.5')^2\right] k_{si} q \quad \text{(Eq D-14)}$$
$$= 4(20.5')^2 (30) k_{si} q$$

The shear panel furthest from the center of rigidity provides the greatest torsional resistance. However, the end panels in the short direction against the exterior walls will not be loaded as heavily as the panels one bay in from the end because the end panels have only half the tributary area as the panel one bay in. Therefore, the panels one bay in from the end will be the most critically loaded because of lateral loads in the short axis and the full width of that bay, and because of its large contribution to torsional resistance. The torsional resistance of the two shear panels that make up the critical short-direction frame, M_{trc}, may be expressed as follows:

$$M_{trc} = \sum_{i=1}^{n} \rho_i^2 k_{si} \theta = 2\left[(3x20.5)^2\right] k_{si} \theta = 2(20.5')^2 (9) k_{si} \theta \quad \text{(Eq D-15)}$$

Equation D-15 shows that the critical short-direction frame provides 3/20 of the total building torsional resistance (Equation D-15 divided by Equation D-14). This ratio can be used to calculate the applied torsion in the critical frame by equating the total accidental torsion, M_{ta}, and torsional resistance from all shear panels, M_{tr}, as follows:

$$M_{trc} = \frac{M_{trc}}{M_{tr}} M_{ta} = \frac{3}{20} M_{ta} \quad \text{(Eq D-16)}$$

The additional shear force due to accidental torsion for the critical frame is now calculated by solving Equation 3-3 for Q_{sic},as follows:

$$Q_{sic} = \frac{M_{trc}}{\rho_c} \quad \text{(Eq D-17)}$$

Values of this additional shear force are given in Table D-3 for each floor level.

Values of seismic story shear in the short direction, V_{xS}, are calculated by modifying Equation C-27 to include the effects of accidental torsion as follows:

$$V_{xS} = \sum_{i=x}^{n} \left(F_i + Q_{sic}\right) \quad \text{(Eq D-18)}$$

D8. LONG-DIRECTION EARTHQUAKE FORCE DEFINITION. The same process is repeated for the definition of earthquake forces in the long direction of the building. These results are summarized in Table D-4. The effects from accidental torsion are not added to the frames in the long direction of the building.

Table D-4. Long Direction Lateral Seismic Force Calculations for the Barracks Building.

Panel Level	Long Direction Total Weight W_L (k-mass)	Seismic Response Coefficient C_s (g)	Long Dir Base Shear V_L (kips)	Height at Floor Level h_{xS} or h_{xL} (ft)	Vertical Distribution Factor C_{vxL}	Number Frames in Long Dir n_L	Long Dir Lateral Seismic Force/frame F_{xL} (kips)	Long Dir Seismic Story Shear V_{xL} (kips)
Roof								
3rd	247			27.042	0.271	2	42.713	42.713
Cumulative	247							
2nd	647			18.583	0.488	2	76.983	119.696
Cumulative	894							
1st	653			9.125	0.241	2	38.110	157.806
Cumulative	1547	0.204	316					

D9. DIAGONAL STRAP DESIGN. From the values of seismic story shear, V_x ($V_x + Q_{si}$ in Equation 3-4) the shear panel diagonal straps are sized according to Equation 3-4. Values of the shear panel design strength, $\phi_t Q_{sy}$ are given in Table D-5. Two identical shear panels are used at each floor level, and applied story shear in the short direction, V_{xS} per shear panel are shown in Table D-5. Trial shear panel dimensions and diagonal strap sizes for each floor level are defined so that the design strength, $\phi_t Q_{sy}$ exceeds the applied story shear, V_{xS} per shear panel, using the spreadsheet program that models Equation 3-4. Table D-5 shows trial shear panel configurations that meet this requirement for each floor of the critical frame in the barracks building example. All diagonal straps require ASTM 653, Grade 33 or Grade 50 steel. Panel dimensions are based on the dimensions given for Shearwall Type "SW-3" (Interior Load-Bearing Walls) of the barracks building drawings (Sheet S-6).

The diagonal straps are the sole lateral-load-resisting element, and as such they determine the story drifts. The elastic deflections, δ_{xe}, at each floor level are calculated as follows:

$$\delta_{xe} = \frac{\delta_{sy}}{Q_{sy}} \frac{V_{xS}}{n_S} \qquad \text{(Eq D-19)}$$

where δ_{sy} is the lateral deflection at diagonal strap yielding given by:

$$\delta_{sy} = \frac{F_{sy}}{E} \left(\frac{H^2 + W^2}{W} \right) \qquad \text{(Eq D-20)}$$

Values of δ_{xe} are given in Table D-5, for the trial diagonal straps at each floor level in the short direction of the building. The design story drifts, Δ are the differences in deflection at the center of mass at the top and bottom of the story under consideration. These deflections are calculated from the elastic deflection, δ_{xe} as follows (from Equation C-29):

$$\Delta = \delta_x = \frac{C_d \delta_{xe}}{I} \qquad \text{(Eq D-21)}$$

Where :

C_d = the deflection amplification factor given in Table D-1 (3.5 for diagonal strap panels).
I = the importance factor given in Table D-1 (1.0 for the barracks building).

Values for the design story drifts are given in Table D-5.

Table D-5. Diagonal Strap Design in the Short Direction.[6]

	Panel Width W (in)	Panel Height H (in)	Strap Faces n_s (#)	Strap Width b_s (in)	Strap Thickness (ga)	Strap Thickness t_s (in)	Strap Initial Lat Stiffness k_s (k/in)	Yield Stress of Strap F_{sy} (ksi)	Strap Lat Yield Capacity Q_{sy} (k)	Design Shear Strength $\phi_t Q_{sy}$ (k)	Lat Defl at Strap Yielding δ_{sy} (in)	Applied Story Shear V_{xS} (kips)	Elastic Lateral Defl δ_{xe} (in)	Defl Amp Factor C_d	Import Factor I	Design Story Drifts Δ (in)	Stability Coeff θ	Allow Story Drifts Δ_a (in)
3rd Floor	132	101.5	1	4	14	0.0747	41	33	7.8	7.4	0.239	6.71	0.205	3.5	1.0	0.718	0.0008	2.03
3rd Floor*	132	101.5	2	4	18	0.0478	53	33	10.0	9.5	0.239	6.71	0.160	3.5	1.0	0.561	0.0006	2.03
2nd Floor	140	113.5	2	6	14	0.0747	112	33	23.0	21.8	0.264	18.52	0.213	3.5	1.0	0.745	0.0015	2.27
1st Floor	140	109.5	2	6	12	0.1046	161	33	32.6	31.0	0.257	24.38	0.192	3.5	1.0	0.672	0.0020	2.19
1st Floor*	140	109.5	2	8	14	0.0747	154	33	31.1	29.5	0.257	24.38	0.201	3.5	1.0	0.705	0.0021	2.19
1st Floor	140	109.5	2	6	14	0.0747	115	50	35.3	33.5	0.389	24.38	0.269	3.5	1.0	0.94	0.0029	2.19

Increases in design story drift, Δ related to P-delta effects are now evaluated. P-delta effects do not need to be considered if the stability coefficient, θ is equal to or less than 0.10. The stability coefficient, θ is defined in Equation C-30 and values are given in Table D-5. These values are well below 0.10, so design story drifts do not need to be increased. Values of design story drifts, Δ must be less than the allowable story drifts, Δ_a given in Table 3-2. For the barracks building this may be expressed as follows (from Table 3-2):

$$\Delta_a = 0.020H \qquad \text{(Eq D-22)}$$

Values of design story drift, Δ and allowable story drift, Δ_a are given in Table D-5 for each floor level for the trial panels in the short direction of the barracks building. The values in Table D-5 show that design story drifts fall below allowable drifts by almost a factor of 3. Therefore these trial sizes meet the drift requirements.

D10. COLUMN DESIGN. Columns are either built-up from studs (Panel A configuration) or are structural tubes (Panel D). The columns built up with cold-formed steel studs must have the studs oriented to form a closed cross-section as shown on Drawings A1 and A2 in Appendix B. Individual studs must be welded to each other with a weld thickness equal to the thickness of the studs. The welds are intermittent, with a length and spacing that will ensure composite behavior of the columns.

Structural tubing columns consist of a single tube, which is a closed section by itself. This column will provide greater moment resistance because of the heavier anchorage detail, and will provide a greater degree of structural redundancy and widening of the shear panel hysteretic performance.

a. Column Applied Loads. Total load applied to the entire building in the short direction is expressed by Equation C-17, where the effects of gravity load and seismic forces are additive and diagonal strap overstrength is accounted for. In this example snow loads, S are zero. This equation can be expressed in terms of the total dead load, D_T, and live load, L, given in Table D-2, as follows:

$$(1.2 + 0.2S_{DS})D_T + 0.5L + \Omega_0 Q_E \qquad \text{(Eq D-23)}$$

The loads expressed in Equation D-23 are now divided between the number of frames that make up the short-direction lateral-load-resisting system. The barracks building has a total of nine such frames. The loads are distributed based on the tributary area of each frame. Because the end bays have only half the tributary area, the loads are divided by the number of frames minus one, or also stated as the number of bays as seen in Table D-6. The vertical load resisting members are the shear panel columns and individual studs, and these are distributed fairly uniformly in plan throughout the building. It is assumed that vertical loads are distributed to these studs in proportion to their area, because of the uniform distribution of columns and individual studs in throughout the building in plan (normally gravity loads would be distributed based on tributary area).

[6] Asterisk designates selected straps.

Trial column stud sizes are selected as summarized in Table D-7. Each frame has two shear panels in the short direction of the building, and each shear panel has two columns so that the 1st, 2nd and 3rd floor columns have four, three and two studs, respectively. This table also summarizes the size of individual studs for the purpose of determining the area of the column studs relative to all other studs. The individual studs include the interior studs inside the shear panels plus all additional individual studs making up the bearing walls in this short-direction frame of the building.

Table D-6. Gravity Load Calculations.

Panel Level	S_{DS} (g)	Total Dead Load D_T=D+EW+IW (kips)	Total Floor Live Load, L (kips)	Short Dir # of bays n_{S-1}	# Studs in Short Dir Col	# Studs in Long Dir Col	Area/ Column Stud A_s (in^2)	Area of Short Dir Col A_{cS} (in^2)	# Ind Stud in Short & Long Dir	Area/ Indiv Stud A_s (in^2)	Area of Indiv & Long Dir Col Studs $A_{I\&cL}$ (in^2)	% Gravity Carried by Short Dir Columns (%)	Gravity /Frame Short Dir GL_{max} (kips)	Gravity /Frame Short Dir GL_{min} (kips)
3rd	0.82	228	0	8	8	8	0.478	3.82	68	0.299	24.16	14%	5.3	2.9
Cumulative		228	0										5.3	2.9
2nd	0.82	507	403	8	12	12	0.747	8.96	68	0.359	33.38	21%	23.6	9.9
Cumulative		736	403										29.0	12.8
1st	0.82	511	403	8	16	16	0.747	11.95	68	0.359	36.36	25%	27.8	11.6
Cumulative		1247	807										56.8	24.4

Table D-7. Trial Stud Sizes and Quantities for One Short-Direction Frame.

Level	Size of Column Studs	Number of Column Studs		Size of Individual Studs	Number of Individual Studs
		Short Direction	Long Direction		
3rd Floor	2” x 6” x 48 mil (18 ga)	8	8	2” x 6” x 30 mil (22 ga)	68
2nd Floor	2” x 6” x 75 mil (14 ga)	12	12	2” x 6” x 36 mil (20 ga)	68
1st Floor	2” x 6” x 75 mil (14 ga)	16	16	2” x 6” x 36 mil (20 ga)	68

Table D-6 summarizes the area calculations based on the trial stud sizes. This table shows that 25, 21, and 14% of the total gravity load in the tributary area of one short-direction frame is carried by the short direction shear wall columns. The remaining gravity loads are carried by individual studs and shear panel column studs in the long direction of the building. These gravity loads are summarized in Table D-6.

The $\Omega_0 Q_E$, term in Equation D-23 accounts for material overstrength in the diagonal straps. The vertical component in the straps will place additional compressive loads in the columns. The total column axial load at the maximum ultimate stress in the diagonal straps, P_{vumax}, is determined from Equation 3-5, and is repeated below:

$$P_{vu\,max} = \frac{GL_{max}}{2} + F_{su\,max} n_s b_s t_s \left(\frac{H}{\sqrt{H^2 + W^2}} \right) \qquad \text{(Eq D-24)}$$

Table D-8 gives values for P_{vumax} for each trial shear wall column at each floor in the short-direction frame of the barracks building.

b. Column Axial Capacity. Table D-8 also presents trial column configurations defined in terms of their yield stress, F_{cy}, column stud or structural tubing material thickness, t_c number of studs per column, panel thickness, b_c and column depth, h_c. The panel thickness is the column width in the out-of-plane direction of the panel and column depth is the column width in the in-plane direction of the panel. Each of the column studs are 6 inches deep with a 2 inch wide flange. They are welded together to form a closed column section and are oriented so that the stud flanges are parallel to the plane of the shear panels (see Panels A1 and A2 in Appendix B). In this orientation, the column depth, h_c is simply the number of studs per column times 2 inches. Table D-9 presents the column capacity calculations. This table gives the column nominal areas, A_c, distance to the extreme fiber, c, in-plane and out-of-plane moments of inertia and radius of gyration. The columns are conservatively assumed to be pinned at their tops and bottom (limited moment resistance when the full axial load is applied) so that the effective length factor, K is 1.0.

Table D-8. Column Design for Cold-Formed Steel Shear Panels – Barracks Example.[7]

	Diagonal Strap Ult Stress F_u (ksi)	Max Ult Strap Stress F_{sumax} (ksi)	Number of Shear Panels /Frame	Max Gravity Load/ Panel GL_{max} (kips)	Column Axial load at Strap Ult P_{vumax} (k)	Column Yield Stress F_{cy} (ksi)	Column Ultimate Stress F_{cu} (ksi)	Column Thickness (ga)	Column Thickness t_c (in)	Number of Studs /Column n	Panel Thickness /Column b_c (in)	Col Stud Flange Width b_f (in)	Column Depth h_c (in)
3rd Floor	45	68	2	2.66	13.6	33	45	16	0.0598	2	6.0	2.0	4.0
3rd Floor*	45	68	2	2.66	17.1	33	45	14	0.0747	2	6.0	2.0	4.0
2nd Floor	45	68	2	14.48	45.3	50	65	14	0.0747	3	6.0	2.0	6.0
1st Floor	45	68	2	28.38	66.4	50	65	12	0.1046	3	6.0	2.0	6.0
1st Floor*	45	68	2	28.38	63.9	50	65	12	0.1046	4	6.0	2.0	8.0
1st Floor	65	81	2	28.38	59.1	46	58		0.1875	1	6.0	6.0	6.0

Table D-9. Column Capacity Calculations for Shear Panels – Barracks Example.[8]

	Nominal Column Area A_c (in^2)	Distance to Extreme Fiber c (in)	In-Plane Mom of Inertia I_x (in^4)	In-Plane Radius of Gyration r_y (in)	Out-of-Plane Mom of Inertia I_y (in^4)	Out-of-Plane Radius of Gyration r_x (in)	Eff Length Factor K	Elastic Flexural Stress F_e (ksi)	λ_c	Nominal Axial Stress F_n (ksi)	Knockout hole dia d_h (in)	Flat Width w (in)	Slenderness factor λ	Eff Width b	Eff Column Area A_e (in^2)	Column Design Strength P (kips)
3rd Floor	1.20	2.00	3.37	1.68	6.25	2.29	1	78	0.65	27.66	1.5	5.761	1.565	2.40	0.794	18.7
3rd Floor*	1.49	2.00	4.16	1.67	7.74	2.28	1	77	0.65	27.61	1.5	5.701	1.239	2.82	1.063	24.9
2nd Floor	2.24	3.21	10.72	2.19	11.61	2.28	1	106	0.69	41.06	1.5	5.701	1.511	2.43	1.508	52.6
1st Floor	3.14	3.21	14.80	2.17	15.99	2.26	1	113	0.67	41.52	1.5	5.582	1.062	3.04	2.34	82.6
1st Floor*	4.18	4.00	32.40	2.78	21.31	2.26	1	122	0.64	42.09	1.5	5.582	1.069	3.02	3.114	111.4
1st Floor	4.27	3.00	23.8	2.36	23.8	2.36	1	133	0.59	39.80	1.5	5.250	0.546	3.75	3.708	125.4

The last row in Table D-8 and D-9 is for a panel with columns made up of 6 x 6 x 3/16 inch structural tubing members (Panel D configuration). The tubing material is ASTM A500 Grade B, with minimum (design) yield stress, F_{cy} and minimum ultimate stress, F_{cu} values of 46 ksi and 58 ksi, respectively. Similar to the column studs, it is assumed that 1.5 inch wide holes will be drilled through the faces of the column that are out-of-plane to the shear panel. These holes are for conduit.

The elastic flexural stress, F_e shown in Table D-9 is calculated based on Equation C-39, and, λ_c is calculated based on Equation C-38. The nominal axial stress, F_n is then calculated based on either Equation C-36 or C-37, depending on the value of λ_c.

The effective areas, A_e, of the columns are calculated according to Equation C-41. Values of the terms used to define this area are also given in Table D-9. Finally, the column design strength, P, is calculated according to Equation C-35. Values of P are given in Table D-9 for each trial column. Through an iterative process in the spreadsheet program, trial column configurations were defined where P exceeds the column axial load at the maximum ultimate stress in the diagonal straps, P_{vumax}. From these results the column configurations marked with an asterisk in Table D-8 and D-9 were selected for the three floor levels[9].

[7] Asterisk designates selected columns.
[8] Asterisk designates selected columns.
[9] The second floor panel shown in these tables is not marked with an asterisk because the panel anchors are inadequate, which may require an increase in column stud thickness, t_c.

c. Column Bending and Composite Behavior. The shear panel anchor guidance will provide moment resistance at the column ends, especially when no axial load is applied to the columns. The columns built up from studs must be designed to act as a composite cross-section. Table D-10 gives the intermittent weld length, L (2 inches for each built-up column in Table D-10) and maximum center-to-center intermittent weld spacing, s_{max} needed to ensure composite behavior of the columns. This is based on Equation 3-9. Based on the values of s_{max} given in Table D-10, actual weld spacing is selected that round down to the nearest full inch from the values given in Table D-10. These welds are made between all studs in the column and begin at both ends of the columns.

d. Column Combined Axial and Moment Capacity. The combination of axial and bending load is evaluated for each trial shear panel. For each case, an interaction value is determined according to Equation 3-10. Table D-10 shows that the interaction values, I fall below 1.0 for all columns in this example.

Table D-10. Column Intermittent Weld Design, and Combined Axial and Moment Capacity.

Max Column Shear V_{cm} (kips)	Area on 1 Side of Crit Weld A (in^2)	Distance to Neutral Axis y (in)	Mom of Column Area Q (in^3)	Weld Shear/ Length q (k/in)	Intermittent Weld Length L (in)	Max o.c. Spacing s_{max} (in)	Strap Max Yield Stress F_{symax} (ksi)	Max Est Lat Defl at Strap Yield δ_{symax} (in)	Applied Moment @ δ_{symax} M_a (k-in)	Column Nominal Moment M_{nx} (k-in)	Column Interaction I
1.1	0.60	1.60	1.0	0.3	2.0	14.3	66	0.478	33.7	55.6	0.952
1.4	0.75	1.60	1.2	0.4	2.0	14.3	66	0.478	41.8	68.7	0.954
2.9	1.49	1.61	2.4	0.7	2.0	12.1	66	0.528	100.4	191.9	0.928
4.2	2.09	1.62	3.4	1.0	2.0	11.7	66	0.514	144.4	265.2	0.967
7.4	2.09	2.20	4.6	1.1	2.0	10.7	66	0.514	274.3	405.0	0.983
							75	0.584	236.0	364.9	0.947

e. Column Shear Capacity. The column design shear capacity, V_C is calculated according Equation C-46 for each trial column. The values are shown in the second column of Table D-11. These are below the strap maximum estimated ultimate lateral capacity ($P_{humax} = \Omega_0 Q_E$) calculated according to Equation 3-24, with values given in the third column of Table D-11. Therefore, the additional shear capacity from anchors is needed to resist the maximum lateral force, as shown in Paragraph 12a.

D11. DIAGONAL STRAP-TO-COLUMN CONNECTIONS. Diagonal strap-to-column connections are designed to resist the maximum estimated ultimate force in the strap, P_{su} defined by Equation 3-15. Values of P_{su} are given in the second column of Table D-12 for each panel.

a. Screwed Fastener Connection Design. All screws used in this example are #10 self-tapping hex head screws, as this is the largest practical size that will not interfere with drywall installation. The nominal screw diameter, d for #10 screws is 0.190 inches.[10] The following fastener layout guidance is based on the Screwed Fastener Connection Design guidance in Chapter 3, Paragraph 9a:

[10] AISI Specification Commentary, Table C-E4-1.

Table D-11. Column and Anchor Shear Design.

	Column Shear Strength V_C (kips)	Strap Lat Ult Capacity $P_{humax}=\Omega_0 Q_E$ (kips)	Yield Stress of Angle F_{yA} (ksi)	Angle Thickness t_A (in)	Anchor Shear Strength V_A (kips)	Total Shear Strength V_T (kips)
3rd Floor	4.7	16.0	36	0.250	32.4	69.5
3rd Floor*	11.8	20.5	36	0.250	32.4	76.6
2nd Floor	26.9	47.0	36	0.250	32.4	91.7
1st Floor	37.7	66.7	36	0.250	32.4	102.5
1st Floor*	50.2	63.5	36	0.250	32.4	115.0
1st Floor	62.1	57.4	36	0.500	64.8	191.7

Table D-12. Screwed Connection Design.

	Max Est	Nominal	Diagonal Strap-to-Column Conn								Design	Number
	Ult Strap Force P_{su} (kips)	Screw Dia d (in)	Strap/Col Thickness Ratio (t_2/t_1)	Tilting Eq C-48 P_{ns} (kips)	Bearing$_1$ Eq C-49&51 P_{ns} (kips)	Bearing$_2$ Eq C-50&52 P_{ns} (kips)	Nominal Shear P_{ns} (kips)	Screw head dia d_w (in)	Nominal Pull-over P_{nov} (kips)	Manufacturer's Nom Shear P_{ns} (kips)	Shear /Screw P_s (kips)	Screws /Face n_{screws} (#)
3rd Floor	20.2	0.19	0.80	1.205	1.724	1.380	1.205	0.402	2.027	1.232	0.602	33
3rd Floor*	25.8	0.19	1.56	1.682	1.103	1.724	1.103	0.402	1.297	1.013	0.506	25
2nd Floor	60.5	0.19	1.00	2.430	1.724	2.491	1.724	0.402	2.027	1.242	0.621	49
1st Floor	84.7	0.19	1.00	4.026	2.415	3.488	2.415	0.402	2.838	1.242	0.621	68
1st Floor*	80.7	0.19	1.40	4.026	1.724	3.488	1.724	0.402	2.027	1.242	0.621	65
1st Floor	72.8											

- Minimum distance between centers of fasteners is 3d = 0.57 inches.
- Minimum distance from centers of fasteners to edge of connected part is 3d = 0.57 inches.
- For connections subjected to shear forces in only one direction, the minimum distance from centers of fasteners to the edge of a connected part perpendicular to the force is 1.5d = 0.29 inches.

The design shear and pull-over per screw, P_s shall be calculated according to Equations C-47 through C-53 based on the thicknesses of the connected members. Table D-12 gives the ratio of t_2/t_1 and the resulting design shear per screw as defined by these equations. A screw head diameter, d_w of 0.402 inches[11] is used for the #10 hex head screws.

Finally, the number of screws required at each diagonal strap-to-column connection, n_{screws} is calculated according to Equation 3-16 for each trial panel configuration. These quantities are given in Table D-12.

Ultimate shear values from the manufacturer's data[12], P_u based on the smaller thickness of the members being connected are used to calculate a nominal screw shear strength, P_{ns} according to the following equation:

$$P_{ns} = \frac{P_u}{1.25} \qquad \text{(Eq D-25)}$$

[11] This dimension was measured from #10 hex washer head screws (ITW Buildex Part Number 1129000) used in test panels at USACERL. Measurement was made using a Vernier caliper and the diameter at the base of the washer head was consistently 0.402 inches ± 0.004 inches (10.2 mm ± 0.1 mm).
[12] From ITW Buildex Catalog for #10 fasteners with #3 drill point.

Equation D-25 is not included in the guidance in Chapter 3 or Appendix C because the format of manufacturer's test data is unknown and may need to be evaluated on a case-by-case basis as shown in this example. Table D-12 provides values for this nominal shear strength based on manufacture's fastener shear strength and Equation D-25. If these values are less than other nominal values based on equations C-48 through C-53, these values will control and will be used in Equation C-47 to calculate the design shear per fastener, P_s. Table D-12 presents P_s based on the overall minimum nominal shear or pull over strength. In this example problem the manufacturer's fastener shear strength data controls the nominal fastener shear strength for all panel configurations except for the one shown on the first row of Table D-12.

The number of screws at each diagonal strap-to-column connection, n_{screws}, shown in Table D-12, is very large and the use of larger screws or a welded connection should be considered. Still, each of these connections may be constructed within the overlap area of the strap and column and within the spacing and edge distance requirements given above. The most difficult joint to lay out is the one in the first row, which is based on installing diagonal straps on only one face of the shear panels. The column is 4 inches wide and the strap is also 4 inches wide and is oriented at an angle based on the width, W and height, H of the overall panel given in Table D-5. A layout of the fasteners is selected that will keep the column critical shear plane as close as possible to the track, while maximizing the net area for rupture strength evaluation. A trial layout is shown in Figure D-4. This connection has 5 fasteners at the first row against the track, and 6, 6, 6, 6, 6 and 5 fasteners in the subsequent rows moving away from the joint. These fasteners are spaced at 9/16 inches on center horizontally and ½ inch on center vertically. The other diagonal strap-to-column screwed connections in Table D-12 are laid-out in a similar manner and are shown in Figures D-5 – D-8.

b. Design Rupture Strength Between Fasteners. Figures D-4 through D-8 show the critical diagonal strap rupture surface, for which the rupture strength is calculated. The rupture surface located along the inside edge of the column and along a horizontal plane will be loaded at approximately a 45 degree angle to the rupture surface. Therefore the average of the shear strength and tensile strength expressed by Equations 3-17 and 3-18 are used for determining the design shear/tension strength, VT along this surface as follows:

$$VT = 0.8\phi_{vt}F_uA_{nvt} \qquad \text{(Eq D-26)}$$

Where:

ϕ_{vt} = the shear tensile rupture resistance factor, equal to 0.75
A_{nvt} = the net area subjected to load at approximately 45 degrees.

The design shear/tension, VT and tensile, T rupture strengths are calculated according to Equations D-26 and 3-18, based on a trial layout of the fasteners in each diagonal strap-to-column connection. The use of #10 screws result in a very large number of screws at each connection as seen in Table D-12 and Figures D-4 through D-8. The screw pattern must stay within the spacing and edge distance limitations given above.

When the strap-to-column rupture strength is evaluated based on Equation 3-19, as modified with Equation D-26 the resistance factors in Equations D-26 and 3-18 may be increased to 1.0, because of the ASTM minimum material requirement on F_u/F_y. All of the trial diagonal strap-to-column connections do not meet Equation 3-19, as can be seen by comparing P_{sy} and $(VT + T)n_s$ in Table D-13. The achieved resistance factor (ϕ_a) for Equation D-26 and 3-18 is shown in Table D-13 for each of these connections. This achieved resistance factor is expressed as follows:

$$\phi_a = \frac{P_{sy}}{n_sF_u(0.8A_{nvt} + A_{nt})} \qquad \text{(Eq D-27)}$$

This factor is well below 1.0 for all connections except the first row (3rd Floor case with a diagonal strap on only one face of the panel). The shear panel shown in the first row (Figure D-4) is therefore an unacceptable configuration, and the design shown in the second row (Figure D-5) is selected for the 3rd floor shear panels. For the other shear panels the resistance factors are above 0.75, but are judged to be acceptable because of the ATSM requirement on F_u/F_y.

Table D-13. Screwed Connection Rupture Strength and Welded Connection Design.

	Strap Yield Force P_{sy} (kips)	Tension /Shear Net Area A_{nvt} (in^2)	Tension Net Area A_{nt} (in^2)	Design Rupture Strength $(VT+T)n_s$ (kips)	Achieved Resistance Factor ϕ_a	Fillet Weld Thickness t (in)	Longitudinal Weld Length L (in)	Longitudinal Weld Design Strength P_L (kips)	Long/Trans Weld Length L	Long/Trans Weld Design Strength P_{LT} (kips)	Welded Conn Total Capacity $(P_L+P_{LT})n_s$ (kips)
3rd Floor	9.9	0.218	0.028	6.8	1.082						
3rd Floor*	12.6	0.044	0.134	11.5	0.825						
2nd Floor	29.6	0.153	0.269	26.4	0.840						
1st Floor	41.4	0.312	0.259	34.3	0.906						
1st Floor*	39.4	0.288	0.299	35.7	0.828						
1st Floor	44.8					0.0747	6.25	12.5	8.75	21.4	67.9

c. Welded Connection Design. Figure D-9 shows a trial layout of a welded diagonal strap-to-column connection. All welds in this connection have a thickness, t equal the thickness of the diagonal strap (0.075 inches). This is much less than 0.15 inches, so weld failure through the weld throat (Equation C-57) need not be considered. Details on the strap and column sizing are given in the last row of Tables D-5 and D-8. All welds have a L/t ratio much greater than 25, so that Equation C-55 is used to define the longitudinal weld capacity. The top edge of this connection shown in Figure D-9 is loaded in the longitudinal direction and its design shear strength is defined according to Equation C-55. The diagonal edges at the end of the diagonal strap are loaded close to 45 degrees, so that an average of Equation C-55 and C-56 defines the weld capacity along these edges. Therefore, the longitudinal/transverse design shear strength (P_{LT}) may be expressed as follows:

$$P_{LT} = 0.87\phi t L F_u \qquad \text{(Eq D-28)}$$

Where:

ϕ = 0.58, which is an average of the resistance factors for longitudinal and transverse loading expressed in Equations C-55 and C-56.

Table D-13 gives the weld thickness, length of welds loaded in the longitudinal and longitudinal/transverse directions. Table D-13 also gives the design capacity of the longitudinal, longitudinal/transverse and combined capacity ($(P_L + P_{LT})n_s$) expressed by Equation 3-21, as modified by Equation D-28. Comparing the total shear capacity and strap yield strength, P_{sy} shows that this connection detail meets the requirements of Equation 3-21.

D12. SHEAR PANEL ANCHORS. Panel anchors must be installed on both sides of the shear panel columns. These anchors are installed at both the top and bottom of the columns to anchor the panels to the floor diaphragms both above and below the shear panels. The anchors are needed to provide the required shear, uplift and moment resistance from the eccentric diagonal strap loading of the anchors. The anchors will also provide limited moment resistance that will allow some moment frame action of the columns, providing system redundancy and a widening of the hysteretic load/deflection envelope. The anchors consist of angle iron sections welded to the column, with loose steel plates that are both bolted to the diaphragm using embedded anchor bolts (see Figures D4 through D9).

a. Anchor Shear Capacity. All of the trial columns shown in Table D-8 have insufficient shear capacity by themselves and require additional shear capacity from their anchorage. The anchor angle irons increase the shear capacity. Each angle leg extends beyond the critical shear plane. Figure D-4 shows such an anchorage made up with 6 inch long, L 4 x 4 x ¼ inch angle iron sections welded to both sides of each column. The anchor shear capacity is defined according to Equation 3-22, and the combined column and anchor shear capacity is defined according to Equation

3-23. The column shear capacity, V_c was determined in Paragraph D10e according to Equation C-46. Table D-14 shows the yield stress, width and thickness of the angles used in these anchors, so that the combined shear strength of the columns and angles V_T exceed P_{humax} (Equation 3-23). Table D-11 shows that combined shear strength V_T exceeds P_{humax} for all the trial shear panels.

b. Shear Panel Anchor Angle and Plate Design. The most critical load condition for anchors is when the effects of gravity load and seismic forces counteract each other, as expressed by Equation C-18.

Column-to-angle welds and angle sizes are selected for each trial configuration based on the guidance in Tables 3-3 and 3-4. For each case, the maximum weld thickness and angle thickness is selected. These sizes are shown in Table D-14 and Figures D-4 through D-9. For each trial anchor, a plate must be added as shown in Figures D-4 through D-9 (and Table D-14), to provide adequate uplift resistance. These plates will also add moment resistance. Anchor angles and plates are designed following the requirements of Equation 3-25, as shown in Table D-14 and D-15.

The capacity of the vertical column-to-angle welds at the corner of the columns assume double shear (Equation C-59), because the effective thickness of this grove weld should be at least twice the thickness of the column material. This is because of the curvature of the column corner that the grove weld will fill. The anchor must provide moment resistance for the moment from the eccentric loading of the diagonal strap, accounting for the maximum estimated yield overstrength of the strap ($P_{symax}L_s$). Any moment capacity beyond this is not required (i.e., P_{cb} in Equation 3-31 may equal zero), but will provide beneficial column moment resistance. However, at any load condition at least one column will have little axial load and no diagonal strap load, so that the anchors will provide significant moment resistance to provide some moment frame capacity in the shear panel.

Table D-14. Shear Panel Anchor Angle and Plate Design.

	Min Gravity Load/ Panel	Anchor Uplift @ max Strap Yield	Col/Anchor Weld Thickness	Anchor Angle Yield Stress	Size							Plate Thickness	Angle Moment Capacity
	GL_{min} (kips)	P_{vymax} (kips)	t_w (in)	F_{yA} (ksi)		H_A		W_A		t_A (in)	k (in)	t_p (in)	M_A (k-in)
3rd Floor	1.44	11.3	0.125	36	L	4	x	4	x	0.25	0.625	0.375	9.87
3rd Floor*	1.44	14.7	0.125	36	L	4	x	4	x	0.25	0.625	0.438	12.34
2nd Floor	6.38	34.1	0.125	36	L	4	x	4	x	0.25	0.625	0.563	18.41
1st Floor	12.21	44.9	0.125	36	L	4	x	4	x	0.25	0.625	0.688	26.01
1st Floor*	12.21	42.5	0.125	36	L	4	x	4	x	0.25	0.625	0.625	22.02
1st Floor	12.21	35.3	0.188	36	L	6	x	6	x	0.50	1.000	0.750	39.49

The panels in Rows 3 and 4 fail to meet the requirement of Equation 3-25, as can be seen in Table D-15. Row 3 (2nd Floor shear panels) is the worst case where the column-to-angle weld design strength, P_A is 3% below the applied load, based on the maximum estimated yield stress of the diagonal straps. This shear panel must be redesigned, which can be done by increasing the thickness of the column material as the strength of the welds are directly proportional to the thickness of the thinner material (see Equations C-56 and C-59). The shear panel shown in Row 5 meets the requirement of Equation 3-25 for the 1st Floor shear panel and this panel is selected for the 1st Floor.

Table D-15. Shear Panel Anchor Angle and Plate Design (continued).

	Distance from Anchor Bolts to: Column Face	Bolt Nut Width	Crit Bending Plane	Tensile Force Avail/angle	Tensile Force/ Angle	Angle Horiz Weld Strength	Angle Vert Weld Strength	Angle Tot Weld Strength
	d_c	W	d_b	P_M	$P_{vymax}/2+P_M$	P_T	P_G	P_A
	(in)	(in)	(in)	(kips)	(kips)	(kips)	(kips)	(kips)
3rd Floor	2.5	1.44	1.16	11.42	17.08	9.69	17.76	27.45
3rd Floor*	2.5	1.44	1.16	14.01	21.34	12.10	11.09	23.19
2nd Floor	2.5	1.63	1.06	17.63	34.66	17.48	16.02	33.50
1st Floor	2.6	1.81	1.09	25.09	47.56	24.48	22.44	46.91
1st Floor*	2.5	1.81	0.97	24.22	45.46	24.48	22.44	46.91
1st Floor	3.5	1.81	1.59	31.90	49.55	39.15	53.83	92.98

Table D-16 presents the anchor (or column) moment capacity as defined by Equation 3-30. Much of this capacity is used to resist the maximum estimated applied moment from the eccentric loading of the diagonal strap ($P_{symax}L_s$). The uplift capacity per angle that remains to resist column bending, P_{cb} should be greater than zero. Table D-16 shows that the panels in Rows 3 and 4 have values slightly below zero. The panel in Row 3 shall be redesigned as stated earlier and the panel in Row 5 is selected for the 1st Floor as stated earlier.

c. Shear Panel Anchor Bolt Design. Finally the anchor bolts that fasten the shear panels to the reinforced concrete beam or slabs are designed. The same detail is used at both the top and bottom of the columns. The anchor bolts are sized based on the bolt shear strength, P_v, tensile strength, P_t and cone failure strength, P_c. Table D-16 and Figures D-4 through D-9 show that two ASTM A-325 anchor bolts are cast into the concrete on both sides of the columns at each anchor, for a total of four bolts per anchor, n_{AB}. The anchor bolts would be positioned with a template before the concrete is cast. Alternatively, the same bolts that anchor the top of one panel could extend through the concrete to anchor the bottom of the panel above.

Table D-16. Anchor Moment and Anchor Bolt Shear Design.

	Column Moment Capacity	Strap Max Yield Strength	Moment Arm of Dia Strap	Angle Uplift for Col Bending	# Anchor Bolts/col	Anchor Bolt Dia	Applied Shear/ Bolt	Bolt Nom Shear Strength	Bolt Shear Design Strength
	M_c	P_{symax}	L_s	P_{cb}	n_{AB}	d_{AB}	P_{hAB}	F_v	P_v
	(k-in)	(kips)	(in)	(kips)	(in)	(in)	(kips)	(ksi)	(kips)
3rd Floor	55.69	19.72	1.60	4.95	4	3/4	4.00	60	19.88
3rd Floor*	68.31	25.24	2.00	3.66	4	3/4	5.12	60	19.88
2nd Floor	121.21	59.16	2.10	-0.44	4	1	11.75	60	35.34
1st Floor	172.51	82.84	2.40	-3.83	4	1 1/8	16.68	60	44.73
1st Floor*	214.93	78.88	2.20	4.66	4	1 1/8	15.89	60	44.73
1st Floor	239.22	67.23	2.20	12.17	4	1 1/8	14.34	60	44.73

The design anchor bolt shear strength, P_v must exceed the applied shear load P_{hAB} (Equation 3-33). Values of P_v, based on Equation 3-34 are given in Table D-16 for each trial panel. In every case these values exceed P_{hAB}. The design tensile strength, P_t and cone failure strength, P_c must exceed the tensile stress per bolt, P_{tAB}. Values for P_t, based on Equation 3-35 and P_c, based on Equation C-60 are given in Table D-17 for each trial panel. In every case these values exceed P_{tAB}. The embedment lengths for the anchor bolts, l_{AB} shown in Table D-17 are very large. If possible the same anchor bolts should rather extend through the concrete beam or slab to the shear panel above or below, thus anchoring the anchor bolts. The minimum edge distance, m to prevent side cone concrete failure is defined based on Equation C-63 and values are given in the last column of Table D-17. Figures D-4 through D-9 show the trial anchor design for each row in Tables D-14 through D-17. For each floor level the asterisk indicates the selected panel design.

Table D-17. Anchor Bolt Tensile and Cone Failure Design.

	Bolt Nom Tensile Strength F_t (ksi)	Bolt Design Strength P_t (kips)	Tensile Force/ Bolt P_{tAB} (kips)	Out-of-plane Space btw Bolts d_{c-c} (in)	Anchor Bolt Embedment Length l_{AB} (in)	Stress Cone Area A_c (in2)	Concrete Compressive Strength f'_c (psi)	Cone Design Strength P_c (kips)	Min Edge Distance m (in)
3rd Floor	90	29.82	22.06	3.5	6.0	110	4000	23.58	3.31
3rd Floor*	90	29.82	27.57	3.5	7.0	143	4000	30.86	3.31
2nd Floor	90	53.01	44.77	3.5	9.0	224	4000	48.27	4.42
1st Floor	90	67.10	67.01	3.0	11.0	315	4000	67.84	4.97
1st Floor*	90	67.10	58.73	3.0	10.0	265	5000	63.61	4.70
1st Floor	90	67.10	56.99	3.0	10.0	265	5000	63.61	4.70

D13. SUMMARY OF EXAMPLE DESIGN PROBLEM RESULTS. The trial shear panel for the 2nd Floor of the barracks building must be re-designed for the column-to-anchor weld detail. Figures D-5 and D-8 show the details for the selected panels. Details for the entire panels are given in Tables D-5 and D-8 through D-17.

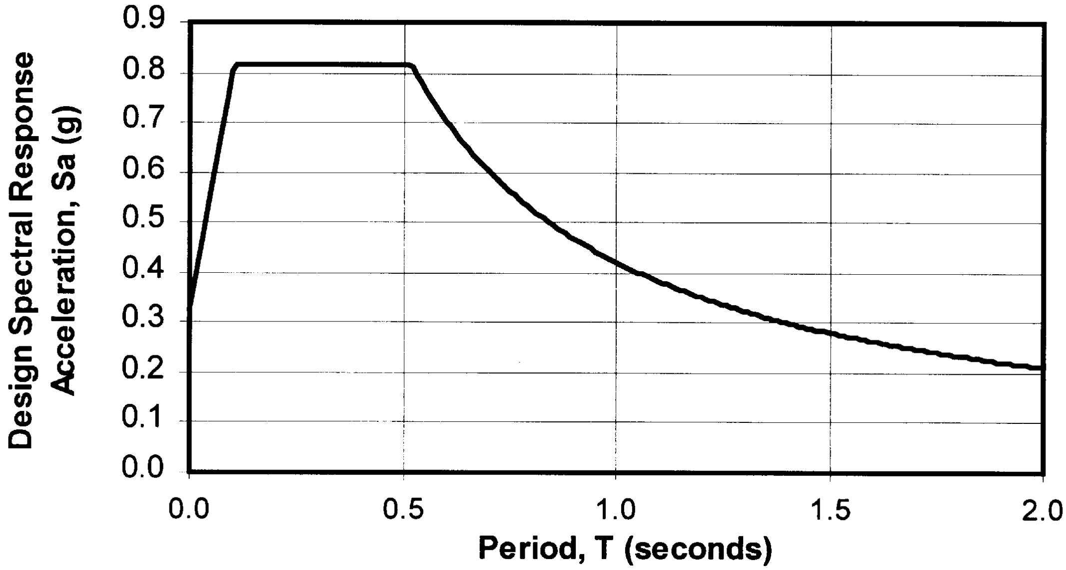


Figure D-1. Design response spectrum for Fort Lewis, Washington barracks building.

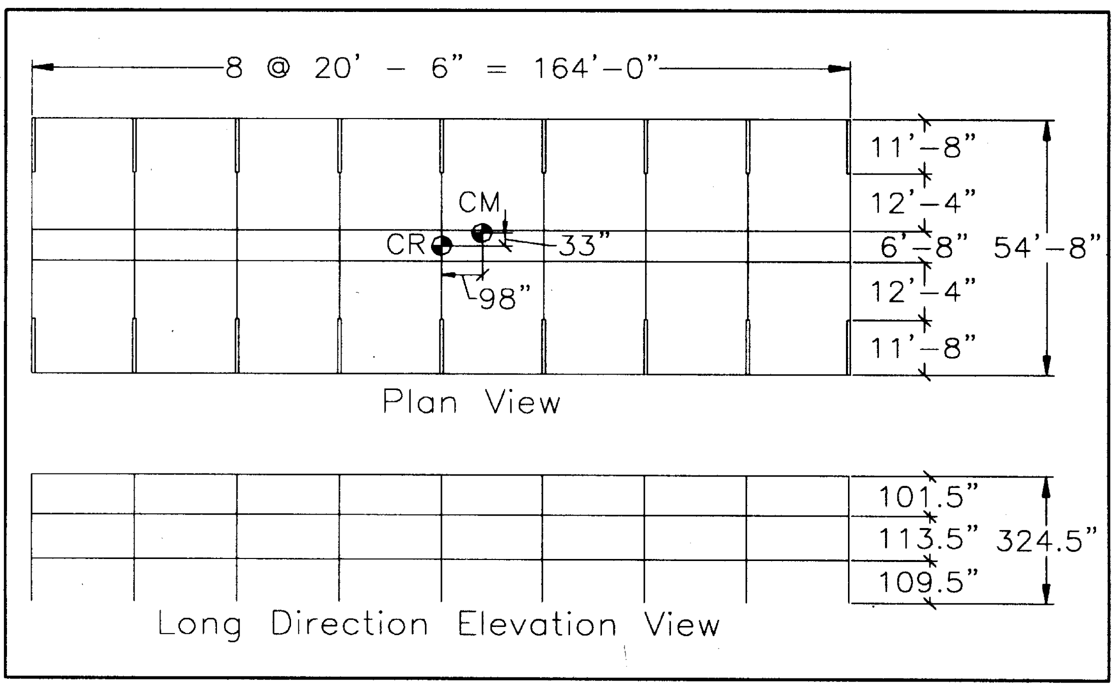


Figure D-2. Schematic drawing of barracks building example.

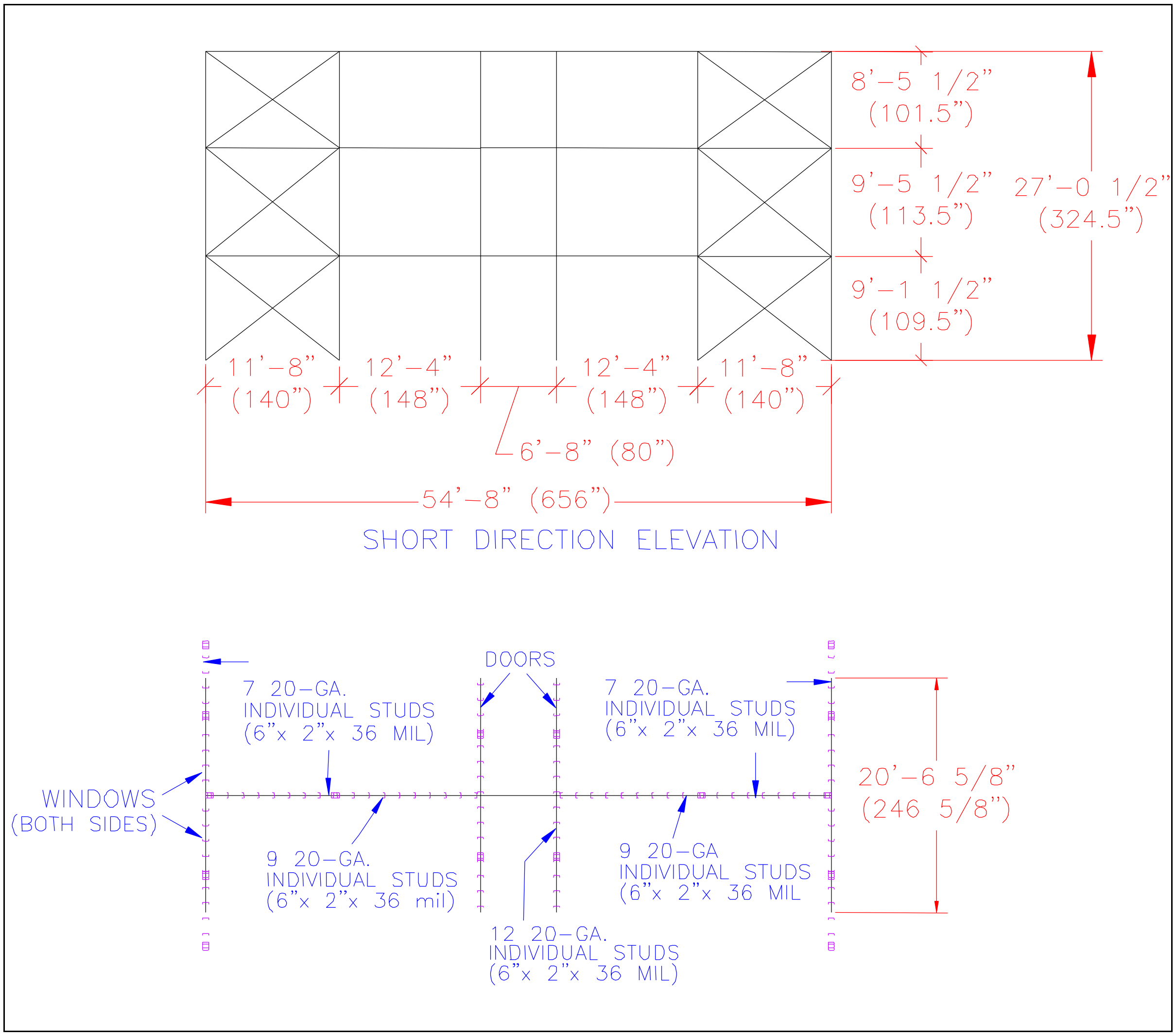


Figure D-3. Barracks building short direction elevation and plan views.

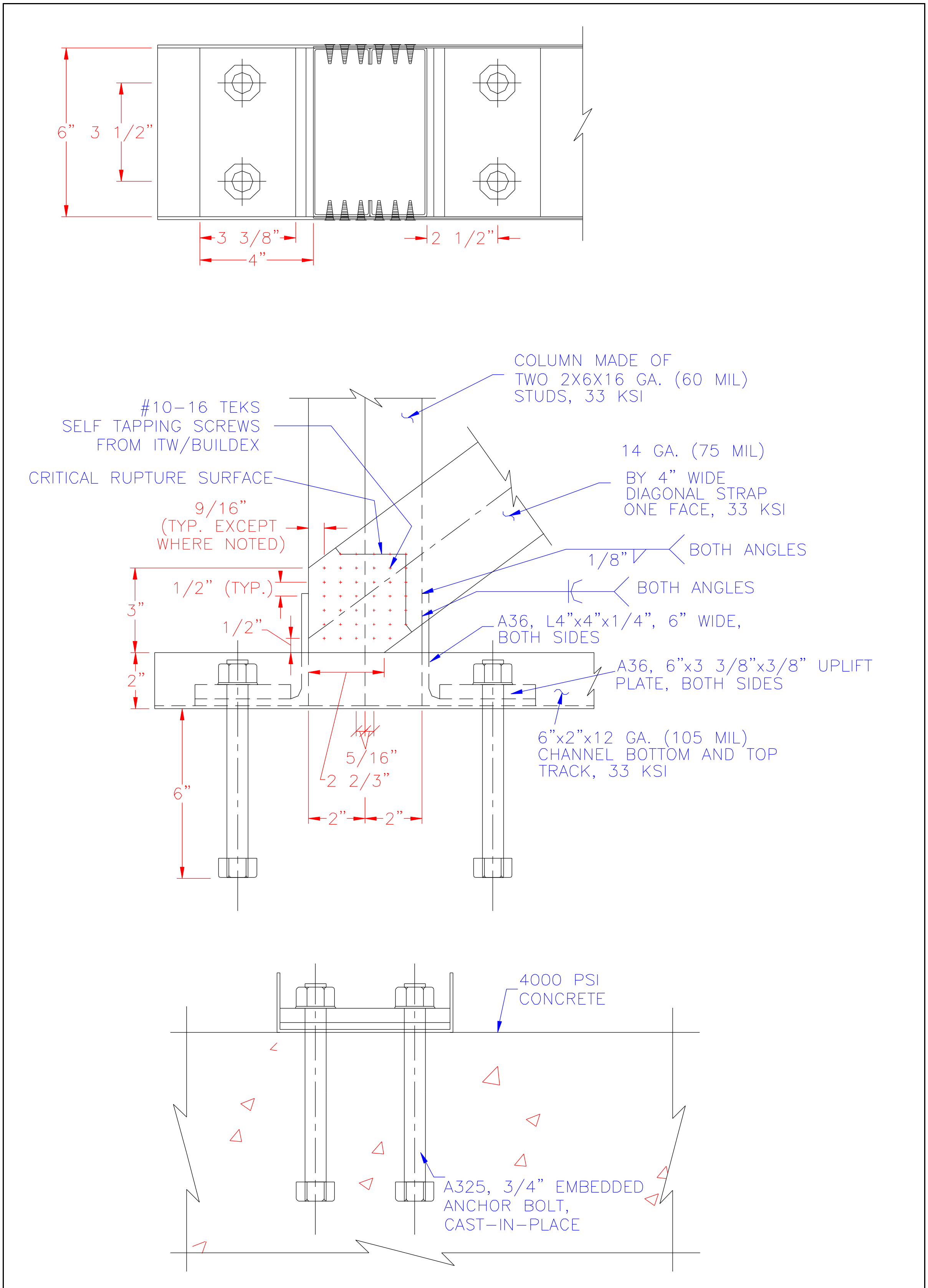


Figure D-4. Example connection/anchorage detail – 1st row of Tables D-5, D-8 – D-17.

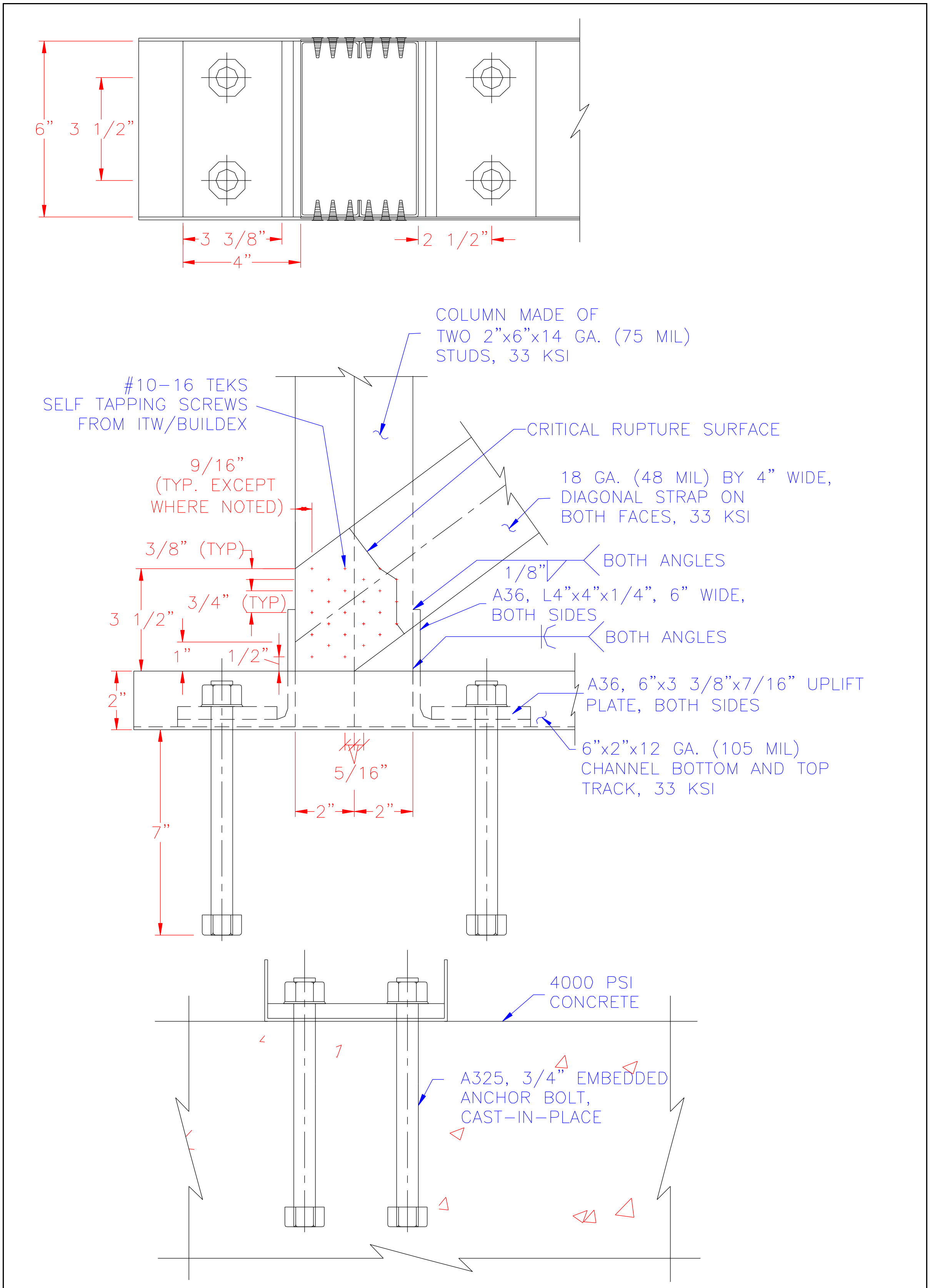


Figure D-5. Example connection/anchorage detail – 2nd row of Tables D-5, D-8 - D-17.

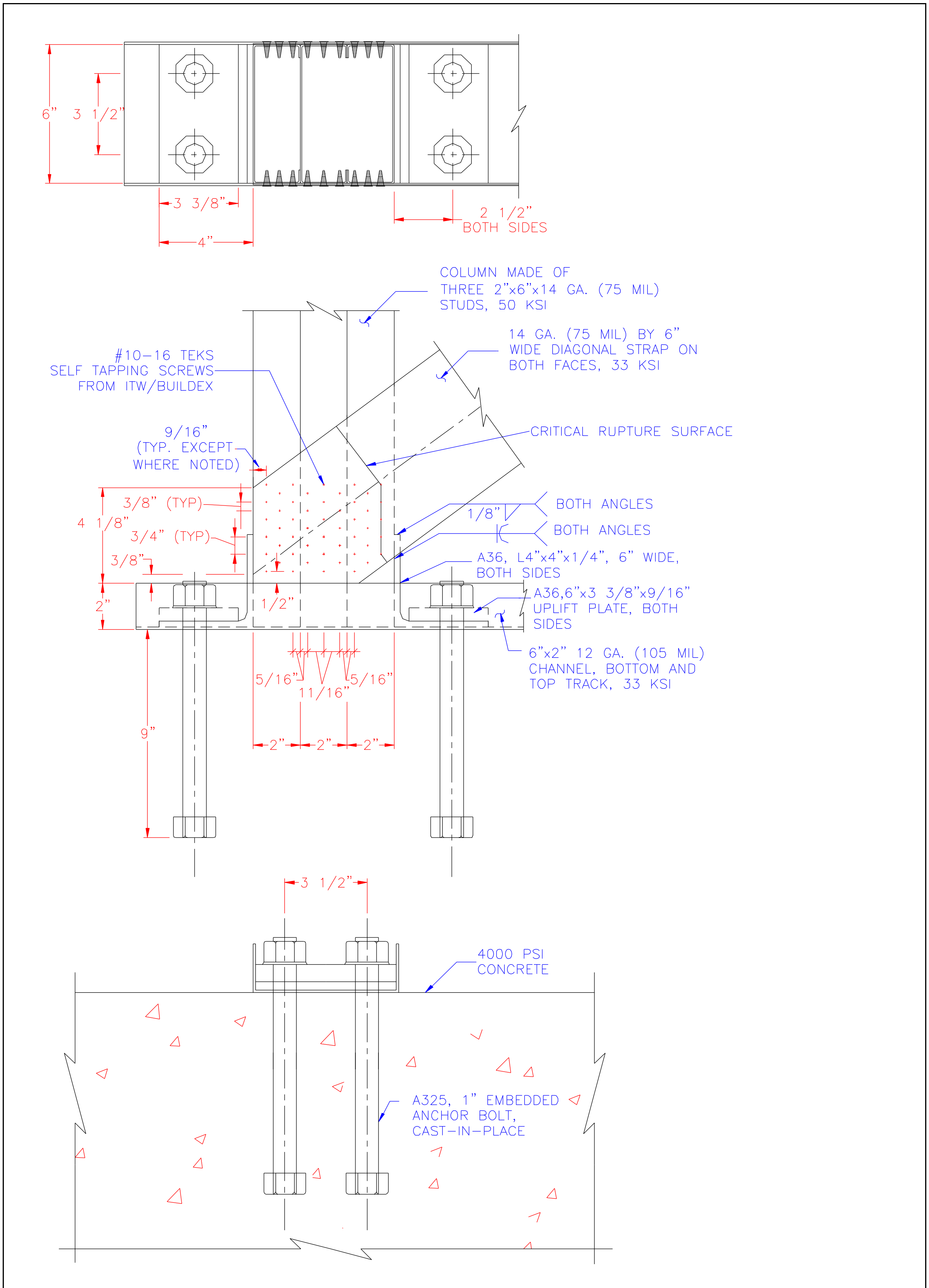


Figure D-6. Example connection/anchorage detail – 3rd row of Tables D-5, D-8 - D.17.

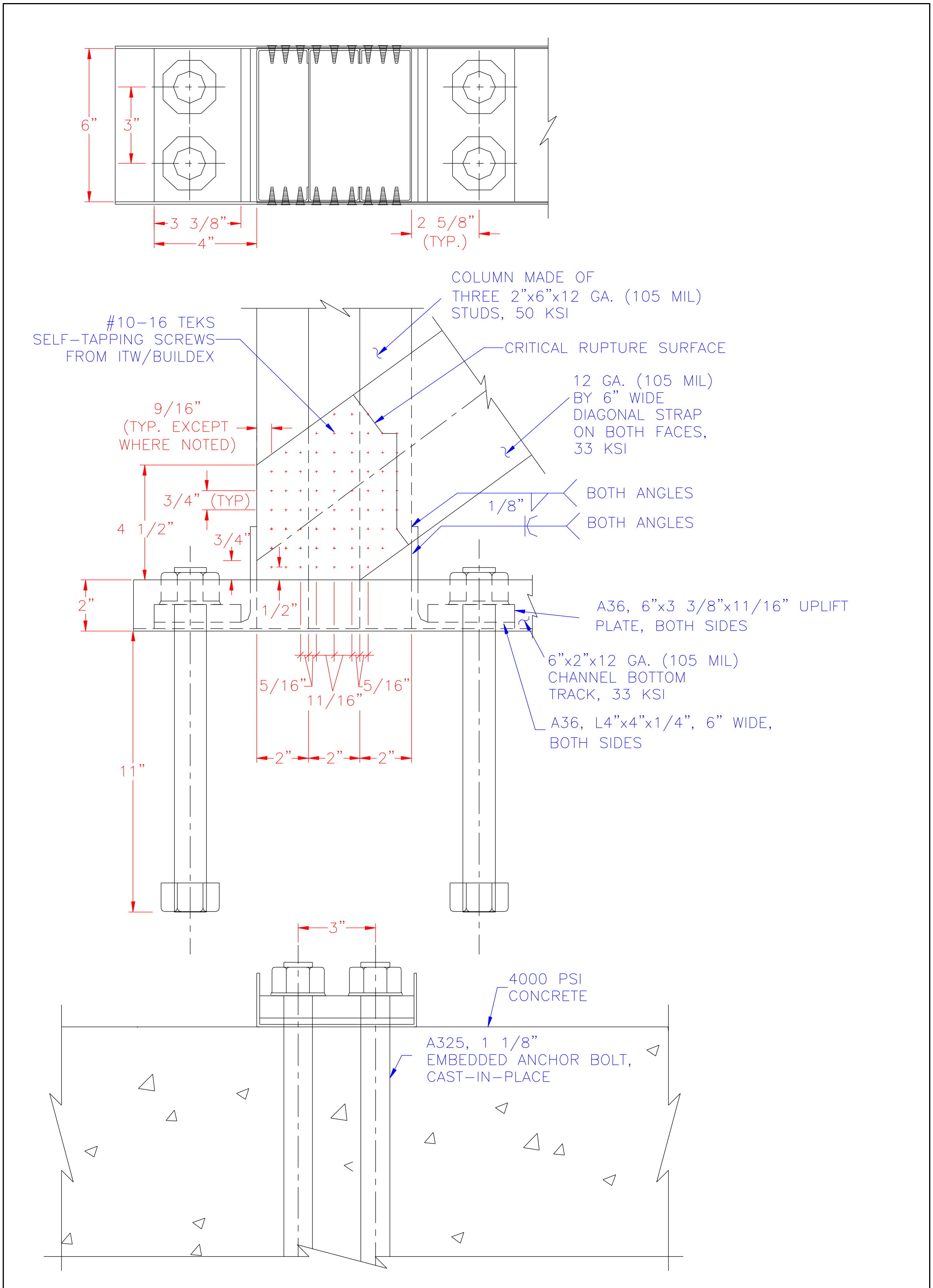


Figure D-7. Example connection/anchorage detail - 4th row of Tables D-5, D-8 – D-17.

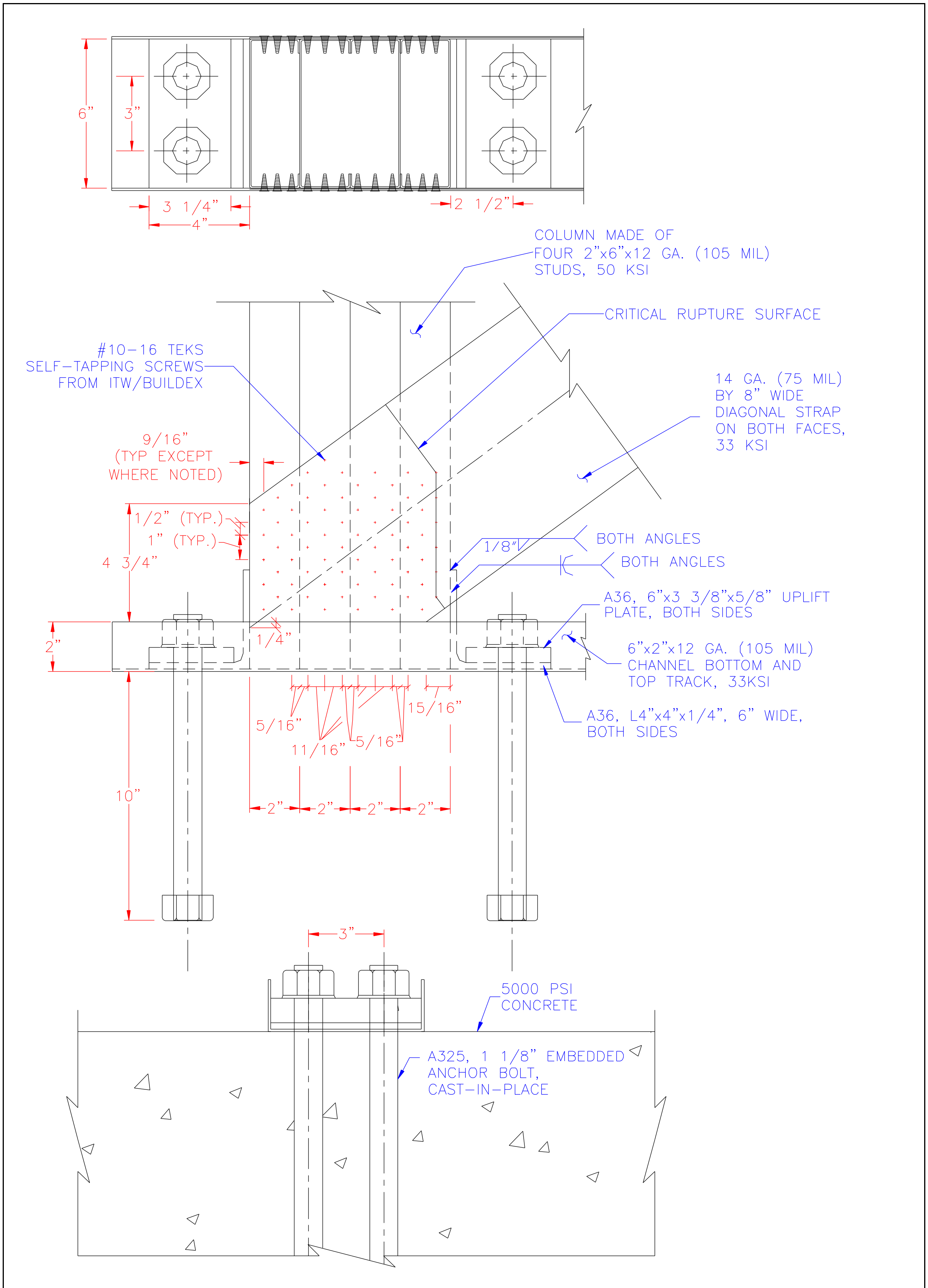


Figure D-8. Example connection/anchorage detail – 5th row of Tables D-5, D-8 – D-17.

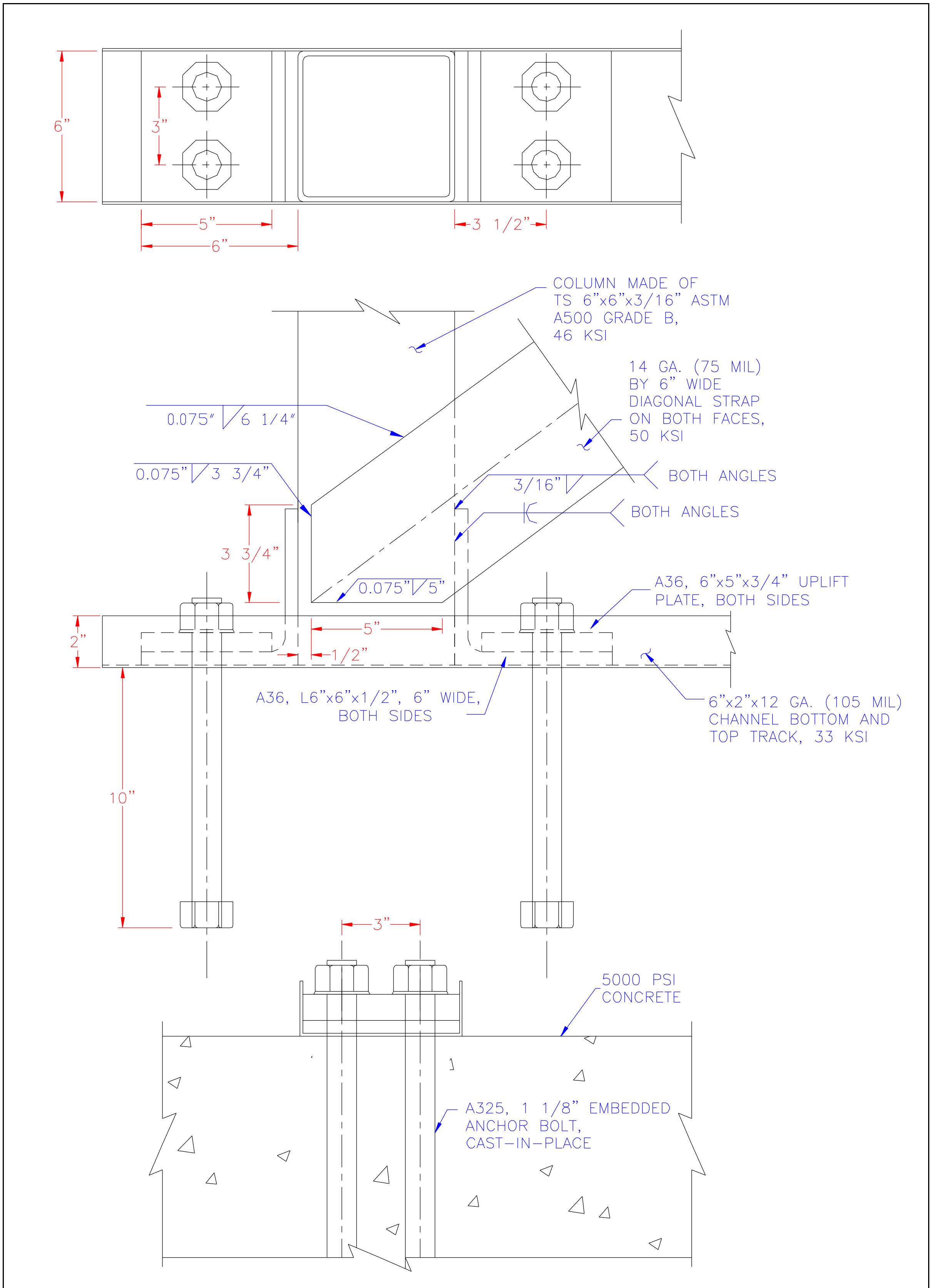


Figure D-9. Example connection/anchorage detail - 6th row of Tables D-5, D-8 - D-17.

APPENDIX E

PROTOTYPE SHEAR PANELS FOR COLD-FORMED STEEL SEISMIC DESIGN
(English I-P units only)

E1. INTRODUCTION. This appendix provides tabular data for the selection of possible prototype shear panels that may be used in the seismic design of cold-formed steel structures. These panels were developed for an example problem presented in Appendix D using the design guidance presented in Chapter 3 and Appendix C. Each shear panel given in Table E-1 is defined in Figures D5, D8 and D9 as indicated in Table E-1.

E2. DEFINITION OF TERMS. The prototype shear panels given in Table E-1 shall be used based on the following definition of terms:

V_x = the maximum story shear per shear panel, based on Equation C-27 in Appendix C.
GL_{max} = the maximum gravity load per shear panel, based on Equation C-17 in Appendix C.
GL_{min} = the minimum gravity load per shear panel, based on Equation C-18 in Appendix C.

E3. PROTOTYPE PANEL LOAD TABLE. Table E-1 provides the tabular data needed to select prototype shear panels.

Table E-1. Prototype Shear Panel Load Capacities.

	V_x (kips)	GL_{max} (kips)	GL_{min} (kips)
Figure D5	9.5	6.0	-13.2
Figure D8	29.5	34.1	-6.5
Figure D9	33.5	44.1	-36.5

APPENDIX F

SEISMIC QUALIFICATION PROCEDURE AND ACCEPTANCE CRITERIA FOR OTHER SHEAR PANEL CONFIGURATIONS

F1. SCOPE. This appendix presents the test procedure, acceptance criteria and documentation requirements needed to demonstrate the acceptability of cold-formed steel shear panel configurations different than the specific system defined in Chapter 3. Acceptable panels are limited to cold-formed steel shear panels with diagonal straps or full panel sheets as the lateral load resisting elements. The columns shall be constructed with cold-formed or hot-rolled structural steel. This is for the qualification of a prototype of the specific panel that will be used in construction. Qualification requires the testing of three specimens. All panel tests shall represent full panel system tests of all the panel components including connections, and anchors.

F2. COUPON TESTS OF ALL TEST PANEL MATERIALS. Coupon tests shall be performed on all materials that may contribute to the structural performance of the test panels. At least three coupons shall be tested from each lot of each type of material. Coupons shall be prepared and tested following the provisions of ASTM A 370. Materials that contribute to the ductility of the shear panels shall have a total elongation of at least 10% for a two-inch long gage length. All coupon test results shall be plotted in a test report, in terms of stress versus strain. All coupon test results shall also be summarized in a table in the format shown in Table F-1. The data in this table shall be the average value of the three or more coupons of the particular component.

Table F-1. Format for Tabular Coupon Test Results.

Structural Component of Coupon	Design Yield Stress (MPa or ksi)	0.2% Offset* Yield Strain (mm/mm)	0.2% Offset * Yield Stress (MPa or ksi)	Maximum Load Strain (mm/mm)	Maximum Stress (MPa or ksi)	Max Stress / 0.2% Offset Yield Stress
Component #1						
Component #2						

* See USACERL TR Chapter 4 for definitions of 0.2% offset yield strain and stress

F3. COUPON TEST OF ALL FIELD PANEL MATERIALS. Coupon tests shall be performed on all materials that contribute to the structural performance of the field panels. The field panels shall be identical to the prototype-tested panels. At least three coupons of each material shall be tested. Coupons shall be prepared and tested following the provisions of ASTM A 370. Materials that contribute to the ductility of the shear panels shall have a total elongation of at least 10% for a two-inch long gage length. All coupon test results shall be plotted in a test report, in terms of stress versus strain. All coupon test results shall also be summarized in a table in the format shown in Table F-1. The data in this table shall be the average value of the three or more coupons of the particular component. The field diagonal straps or full panel sheets shall have a coupon yield stress (0.2% offset) not greater than 5% above or not less than 10% below the test panel coupon yield stress (0.2% offset). The field material coupons for all other structural elements shall have coupon yield stress (0.2% offset) not less than the test panel coupon yield stress (0.2% offset).

F4. TEST CONFIGURATION. Full-scale test panels shall be tested with both monotonic (pushover in one direction) and cyclic loading. The panels shall be anchored to a base beam and top beam in a manner representative of the field installation. The base beam shall resist any slippage, out-of-plane movement or rotation in any direction. Vertical load shall be applied to the shear panel through the top beam, at a level representative of potential gravity loads in the field. The amount of vertical load applied should consider the worst case condition for the most vulnerable panel components. For example, the minimal vertical load may provide the most severe loading for the anchors, while the maximum vertical would provide the worst case loading for column buckling. This vertical load shall be held constant throughout each test. The top beam shall be held horizontal during all tests, as this represents the field conditions when the panel is assembled in a building. Figure F-1 shows the test configuration and instrumentation plan for shear panels tested at USACERL in order to illustrate the load configuration. In the USACERL tests, stroke control was used to keep the two vertical actuators at the same length, which held the top beam horizontal. The combined vertical force was held constant manually.

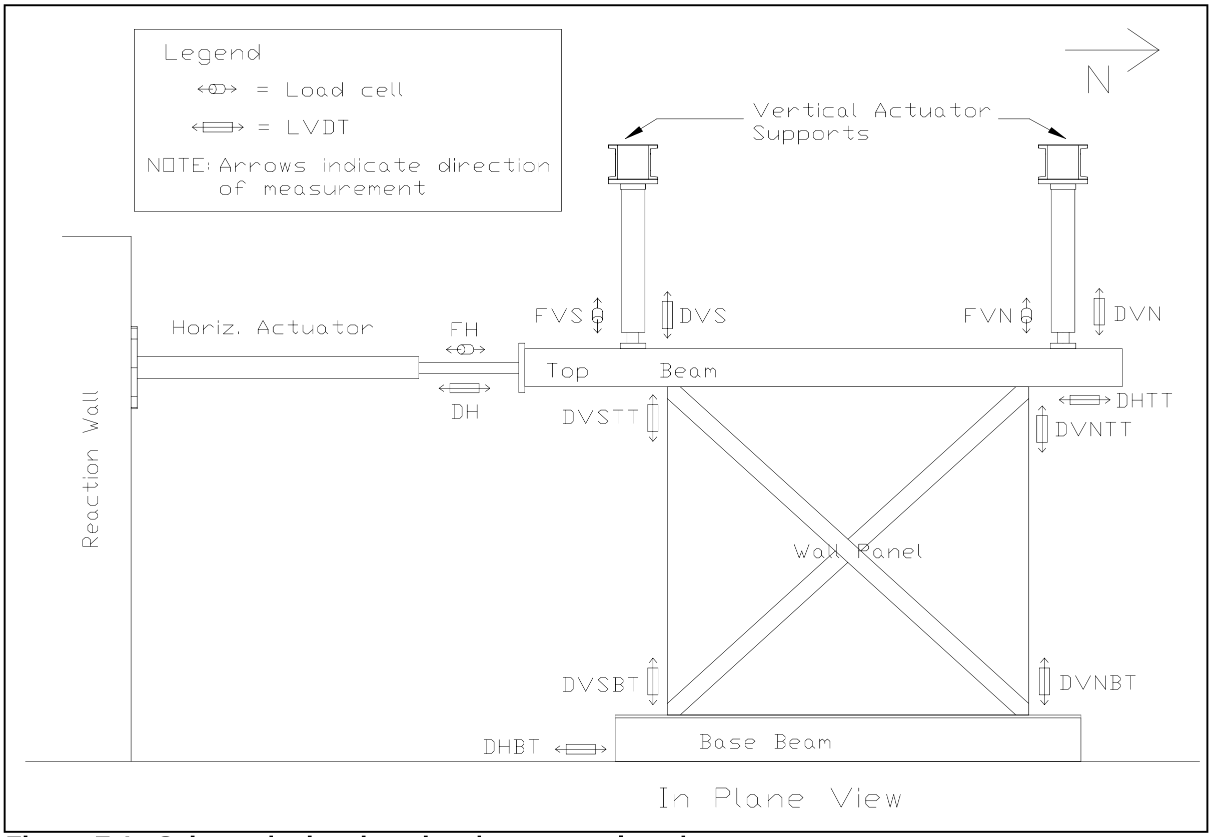


Figure F-1. Schematic drawing showing sensor locations.

F5. INSTRUMENTATION. Table F-2 defines the instrumentation required for all shear panel tests. Figure F-1 shows the location and orientation of all sensors. Table F-2 describes the purpose of each sensor. The purpose of most gages is to ensure that no unwanted motion takes place and for test control. The only data used in reporting panel performance are the 1st, 2nd, 3rd and 4th channels in Table F-2. The vertical actuator force measurements (FVS and FVN in Table F-2 and Figure F-1) are required to defined total shear force when deflections reach large amplitudes, at which point the horizontal components of these forces become significant. This total shear force, TSF is determined as follows:

$$TSF = FH - TVF\left\{\sin\left[\arctan\left(\frac{DH}{L}\right)\right]\right\} \qquad \text{(Eq F-1)}$$

Where:

FH = the measured horizontal actuator force (see Table F-2 or Figure F-1).
TVF = the total vertical actuator force, equal to FVS plus FVN (Table F-2 or Figure F-1).
DH = the measured horizontal displacement (Table F-2 or Figure F-1).
L = the length of the vertical actuators, with the vertical load applied, but not horizontal displacement.

Table F-2. Cold-Formed Steel Shear Panel Instrumentation.

Channel #	Sensor Type	Measurement, Direction, Location and Symbol	Purpose
1	Load cell	Force Horizontal, FH	Horizontal actuator load measurement
2	LVDT	Deflection Horizontal, DH	Horizontal deflection, shear panel deformation
3	Load cell	Force Vertical South, FVS	Manual vertical load control (25k total load w/#5)
4	LVDT	Deflection Vertical South, DVS	Stroke (tied to #6)
5	Load cell	Force Vertical North, FVN	Load (summed with #3, for 25k total load)
6	LVDT	Deflection Vertical North, DVN	Controlled by #4 stroke feedback
7	LVDT	Defl Horiz Bot Track, DHBT	To ensure no slippage
8	LVDT	Defl Vert South Bot Track, DVSBT	To ensure no uplift
9	LVDT	Defl Vert North Bot Track, DVNBT	To ensure no uplift
10	LRDG* (20")	Defl Horiz Top Track, DHTT	Check for shear panel deformation - same as #2
11	LRDG (10")	Defl Vert South Top Track, DVSTT	Vertical panel/column deformation & rotation check
12	LRDG (10")	Defl Vert North Top Track, DVNTT	Vertical panel/column deformation & rotation check

Note: * Linear Resistance Deflection Gauge, often called a Yo-Yo Gauge.

F6. TEST REQUIREMENTS. For each shear panel qualified, three specimens shall be fabricated and tested. This assumes only minor variation in panel performance for a given shear panel. If large variations occur more than three specimens shall be tested and a statistical evaluation of panel performance may be required. For panels with minor variation, one specimen shall be tested monotonically and two shall be tested cyclically as defined below. All tests, both monotonic and cyclic shall use stroke control, loading the panels laterally at a constant displacement per minute. The vertical load shall be held constant and the top beam shall be held horizontal throughout each test as described in Paragraph F4, Test Configuration. Both monotonic and cyclic tests shall be conducted up to deflections that cause ultimate failure of the shear panels, or reach the limits of the test equipment, but shall not be less than 10 times the lateral yield displacement of the test panel, δ_y. These are very large deflections, well beyond acceptable drift limits, but are needed to ensure that brittle failures (sudden loss of lateral or vertical load carrying capacity) do not occur near the useful deflection range of the panel.

a. Monotonic Test Protocol. A single specimen of each shear panel shall be loaded in one direction (monotonic) at a constant stroke rate that is slow enough to allow careful observation of panel performance and failure progression[1]. These observations shall include documentation of panel behavior through a log of observations with respect to displacement and photographs. Load versus deflection (TSF versus DH) shall be plotted to determine the measured lateral yield displacement δ_y. This value shall be used in defining the cyclic test protocol.

b. Cyclic Test Protocol. A minimum of two specimens of each panel configuration shall be loaded cyclically at a constant stroke rate that is slow enough to allow careful observation of panel performance and failure progression[2]. These observations shall include documentation of panel behavior through a log of observations with respect to displacement and photographs. Load versus deflection (TSF versus DH) shall be plotted to create load/deflection hysteretic envelopes. The cyclic load protocol follows a standard method, so that test results may be compared with cyclic test results of other systems. The protocol defined here is similar to SAC2[3] guidelines that have been modified to scale to the lateral yield deflection as described in ATC-24[4]. The SAC recommended loading histories call for loading with a deformation parameter based on interstory drift angle, θ defined as

[1] Monotonic tests conducted at USACERL used a stroke rate of 0.5 inches per minute.
[2] Cyclic tests conducted at USACERL used a stroke rate of 3 and 6 inches per minute. The faster stroke rate was used for panels tested cyclically beyond 10 inches (20 inches peak to peak).
[3] SAC Testing Programs and Loading Histories, unpublished guidance, 1997.
[4] Applied Technical Council (ATC) 24, Guidelines for Cyclic Seismic Testing of Components of Steel Structures, 1992.

interstory height over interstory displacement. The commentary to the SAC document explains that the interstory drift angle of 0.005 radians corresponds to a conservative estimate of the value that would cause yield deformation. Therefore, the load protocol defined by SAC in terms of drift angle are scaled to the measured lateral yield deflection, δ_y to define the cyclic test steps as defined in Table F-3. This protocol calls for a set number of cycles at each of the deformation amplitudes shown in Table F-3. This protocol is illustrated by the deformation time history shown in Figure F-2, which is based on a lateral yield deflection, δ_y of 0.4 inches and stroke rate of 6 inches per minute.

Table F-3.Cyclic Test Load Protocol.

Load Step #	SAC-2		Modified SAC Amplitude
	Number of Cycles, n	Peak Deformation, θ (radians)	
1	6	0.00375	$0.75\delta_y$
2	6	0.005	$1.0\delta_y$
3	6	0.0075	$1.5\delta_y$
4	4	0.01	$2\delta_y$
5	2	0.015	$3\delta_y$
6	2	0.02	$4\delta_y$
7	2	0.03	$6\delta_y$
8	2	0.04	$8\delta_y$
9	2	0.05	$10\delta_y$
10	2	0.06	$12\delta_y$
11	2	0.07	$14\delta_y$
12	2	0.08	$16\delta_y$
13	2	0.09	$18\delta_y$
14	2	0.10	$20\delta_y$
15	2	0.11	$22\delta_y$
16	2	0.12	$24\delta_y$
17	2	0.13	$26\delta_y$
18	2	0.14	$28\delta_y$
19	2	0.15	$30\delta_y$
20	2	0.16	$32\delta_y$

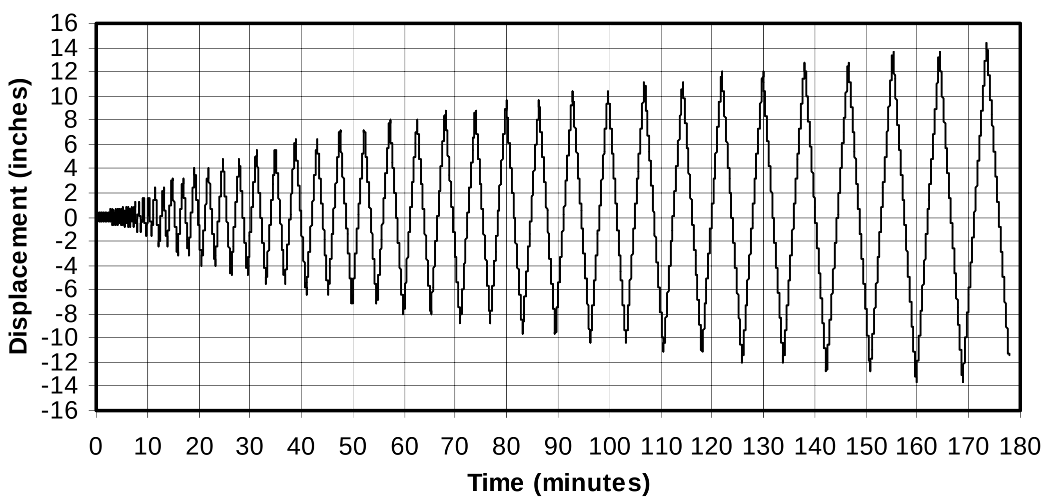


Figure F-2. Modified SAC cyclic test time history with δ_y = 0.4 in and 6 in/min stroke rate.

F7. SHEAR PANEL PERFORMANCE DOCUMENTATION. Shear panel performance from both monotonic and cyclic tests shall be documented in terms of load versus deflection plots (TSF versus DH). Cyclic tests plot load versus deflection to define load versus deflection hysteretic envelopes. Observations of panel performance and failure progression with respect to lateral displacement shall be documented in a tabular or other format. Photographs that document these observations shall be included in the test report. Test results for each specimen tested shall be summarized as indicated in Table F-4. Repeatability of panel performance of a given configuration is critical, so that if only two cyclic tests are conducted the poorest performance of the two shall form the basis for design. Therefore special consideration shall be given to large variations in panel performance, especially failure type or displacement amplitude of each type of failure. Test procedures and results shall be documented in a test report.

Table F-4. Summary of Test Panel Performance.

Test Specimen	Load Type (Monotonic or Cyclic)	Load Rate (mm/min or in/min)	Linear Shear Stiffness (kN/mm) or (kips/inch)	Shear Load at δ_y Deflection (kN or kips)	Shear Deflection at Ultimate Shear Load (mm or inches)	Ultimate Shear Load (kN or kips)

F8. DESIGN GUIDANCE. The measured load versus deflection data shall be used to define the design strength and stiffness of the shear panels. Resistance factors for each loading mechanism shall be defined that recognizes the variation of the shear panel capacity. In other words a panel shear capacity resistance factor, ϕ_v, shall reflect the variability of shear capacity of the tested panels. For example, $\phi_v = 0.9$ if the strength variability is small and mode and displacement of failures are consistent.

The following criteria shall be defined from the shear panel cyclic test data:

1. The panel ductility, μ , is the ultimate lateral deflection without loss of lateral or vertical load capacity, δ_u over yield lateral deflection, δ_y defined as follows:

$$\mu = \frac{\delta_u}{\delta_y} \qquad \text{(Eq F-2)}$$

2. The panel overstrength, Ω[5], which is the maximum measured ultimate lateral panel capacity, Q_u over the yield capacity, Q_y, defined as follows:

$$\Omega = \frac{Q_u}{Q_y} \qquad \text{(Eq F-3)}$$

3. The panel redundancy factor, ρ_1 of the individual shear panel tested[6]. This redundancy can be seen by comparing shear panel load/deflection data with coupon data, to determine if overstrength, Ω is due to strain hardening of the primary load-carrying element or due to the action of a secondary lateral load-resisting element. An example of this would be a panel with diagonal straps acting as the primary element with the columns effectively working to provide a

[5] This should not be confused with the system overstrength factor, Ω_0 defined in FEMA 302, Section 5.2.2 and Table 5.2.2 or TI 809-07, Equation C-16.

[6] This should not be confused with the reliability factor, ρ or ρ_x, which is the extent of structural redundancy in the lateral-force resisting system for an entire story of a building.

significant moment frame. In this case the moment frame would provide redundancy for the shear panel. If the diagonal straps fail, this moment frame capacity would provide lateral resistance for the moment from the P-delta effect of the gravity load. This redundancy is critical to preventing building collapse for a structure whose lateral load resisting system has failed. The panel redundancy factor, ρ_1 is calculated as follows:

$$\rho_1 = \frac{Q_u}{Q_p} = \frac{Q_p + Q_c}{Q_p} \qquad \text{(Eq F-4)}$$

Where:

Q_p = the portion of the shear panel ultimate lateral capacity carried by the primary lateral load resisting element including the effects of strain hardening. For panels with full panel sheet(s) this contribution will increase with increasing deflection due to a widening of the panel tension field. This value can only be reasonably determined by measuring Q_c (as described below) and calculating Q_p as the difference between Q_u and Q_c.

Q_c = the portion of shear panel ultimate lateral capacity carried by the columns acting as moment frames. For panels with full panel sheet(s) this value can only be obtained by testing the same exact panels with the full panel sheets removed. If these tests are not performed for full panel sheet shear panels, Q_c shall be set equal to zero.

4. The width of the cyclic test load/deflection hysteretic envelope. If the hysteretic envelope is significantly pinched (no or very little load resistance away from the peak excursions), much less energy is absorbed by the structural system so that building amplification grows. Pinched hysteretic envelopes occur when the primary lateral load-resisting element is stretched, and there is little redundant capacity from other elements to pick up load, so that little resistance is available away from the peak excursions of the load cycles. Panels with significantly pinched hysteretic envelopes, can experience high acceleration impact loading because the building will be free to sway with little resistance and then suddenly snap the lateral load-resisting element when another peak excursion is reached. This high acceleration snap can cause brittle failures. A shear panel with a great deal of redundancy within the panel, ρ_1 will tend to have a wide hysteretic envelope.

Table F-5 defines the acceptance criteria in terms of μ, Ω and ρ_1 based on data measured in the cyclic panel tests, as defined by Equations F-2 through F-4.

Table F-5. Acceptance Criteria for Shear Panels based on μ, Ω and ρ_1.

Criteria	Acceptance Requirement
Panel Ductility, μ	≥ 10
Panel Overstrength, Ω	≥ 1.3
Panel Redundancy factor, ρ_1	≥ 1.0
Hysteretic Envelope Width	Not Required

Values for the system response modification factor, R system overstrength factor, Ω_0 and deflection amplification factor, C_d, are defined in Table F-6. These values are used in the seismic design guidance defined in TI 809-04 and FEMA 302. Exceptions to this criteria shall require Corps of Engineers Headquarters (CEMP-ET) approval.

Table F-6. Values for R, Ω_0 and C_d.

Factor	Value
System Response Modification Factor, R	4
System Overstrength Factor, Ω_0	2
Deflection Amplification Factor, C_d	3.5

APPENDIX G

MASONRY VENEER / STEEL STUD WALLS (NONBEARING CONSTRUCTION)

List of Drawings

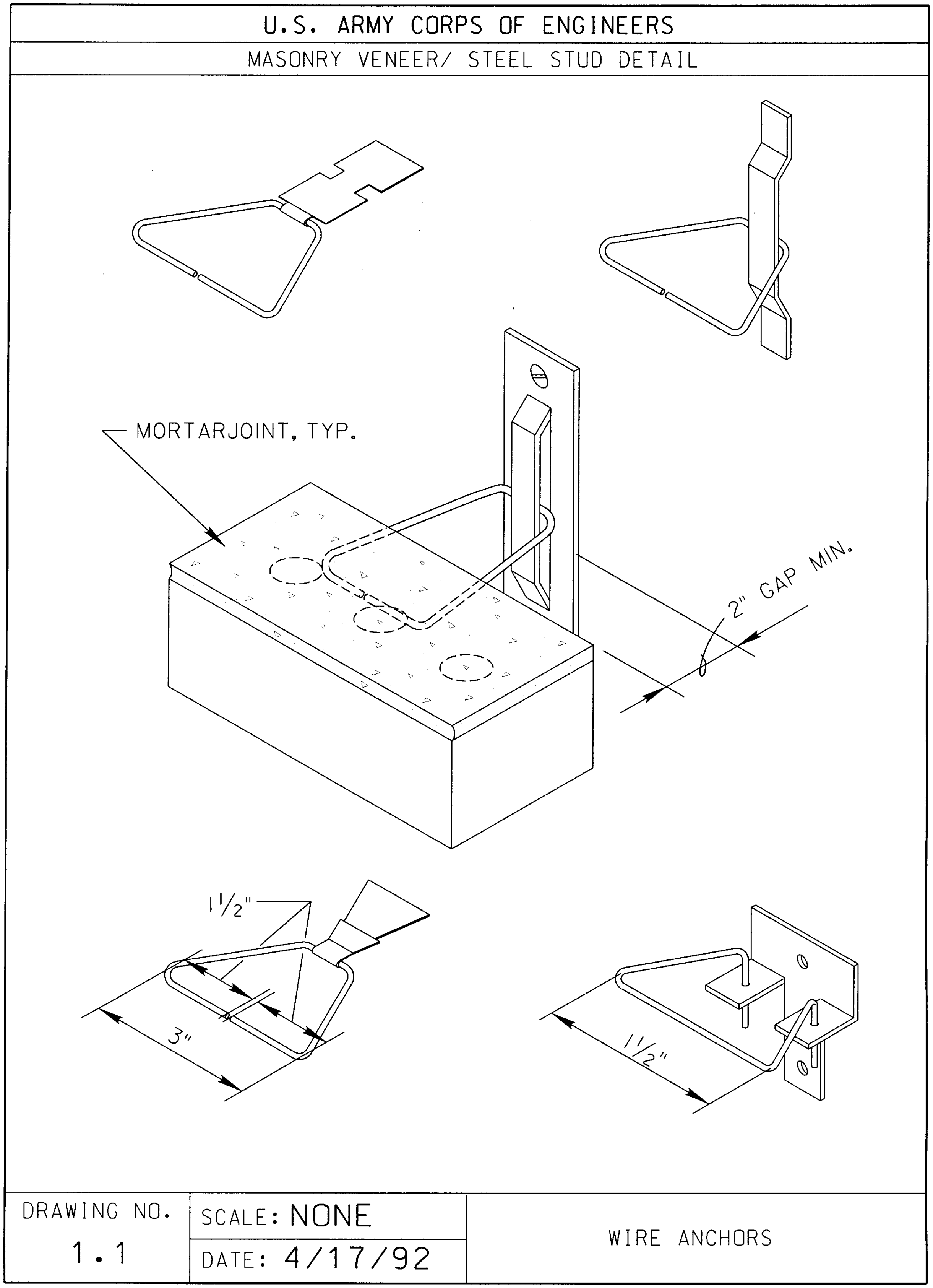
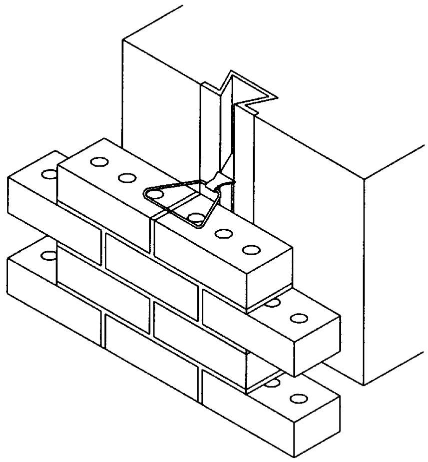


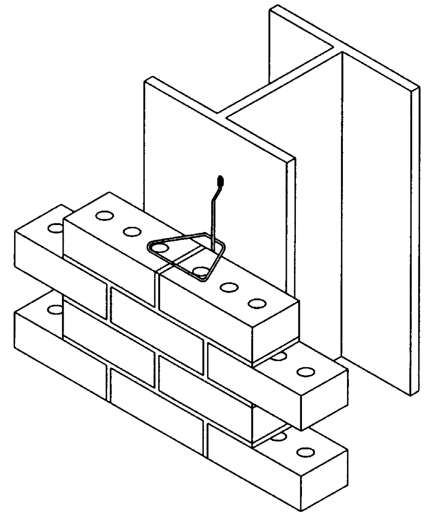
Figure G-1. Wire Anchors

U.S. ARMY CORPS OF ENGINEERS

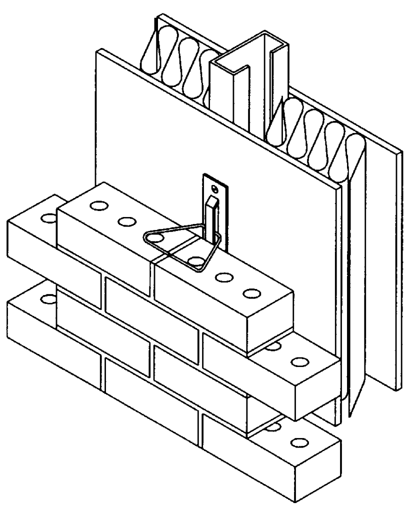
MASONRY VENEER/ STEEL STUD DETAIL



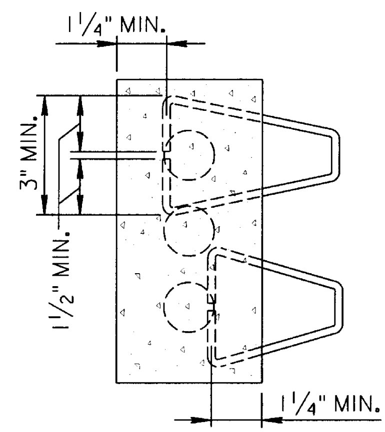
A) CONCRETE BACK-UP



B) STRUCTURAL STEEL BACK-UP



C) STEEL STUD BACK-UP



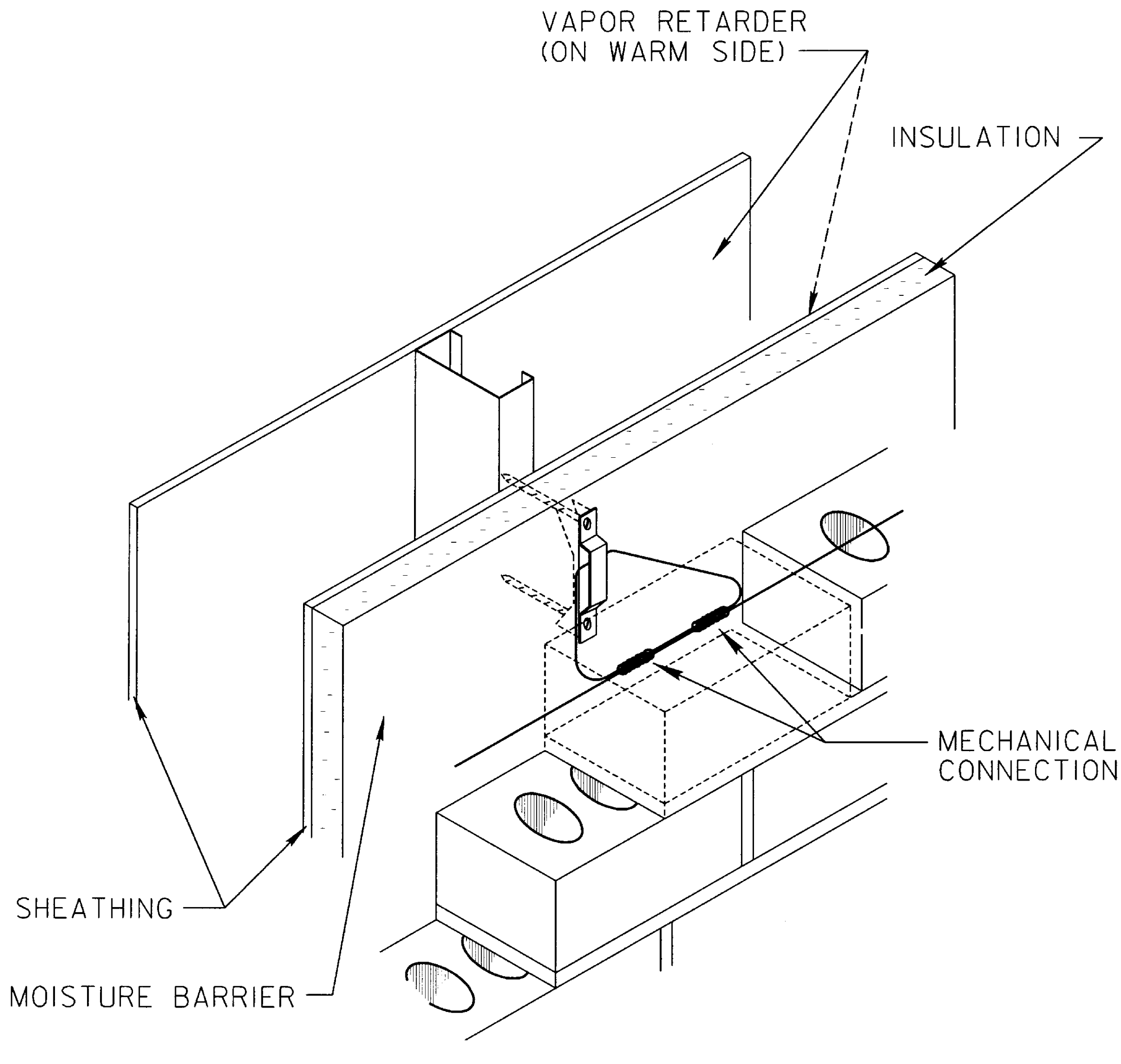
TIE PLACEMENT DETAIL

DRAWING NO.	SCALE: NONE	WIRE ANCHOR, DETAILS
1.2	DATE: 4/17/92	

Figure G-2. Wire Anchor, Details

U.S. ARMY CORPS OF ENGINEERS

MASONRY VENEER/ STEEL STUD DETAIL



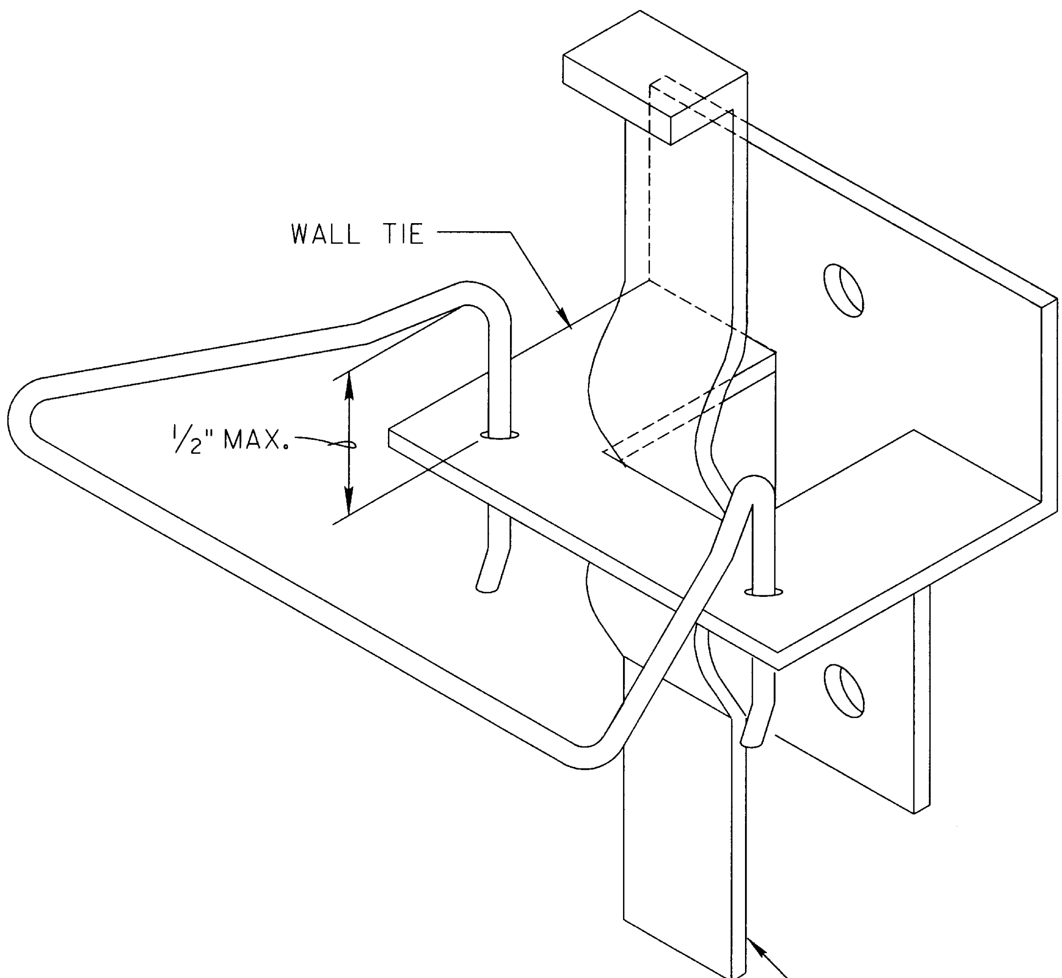
JOINT REINFORCEMENT IN THE SAME COURSE AS THE ANCHOR WILL BE MECHANICALLY CONNECTED AND MAY DIFFER FROM THAT SHOWN.

DRAWING NO.	SCALE: NONE	WIRE ANCHOR WITH CONTINUOUS BRICK JOINT REINFORCEMENT
1.3	DATE: 4/17/92	

Figure G-3. Wire Anchor with Continuous Brick Joint Reinforcement

U.S. ARMY CORPS OF ENGINEERS

MASONRY VENEER/ STEEL STUD DETAIL



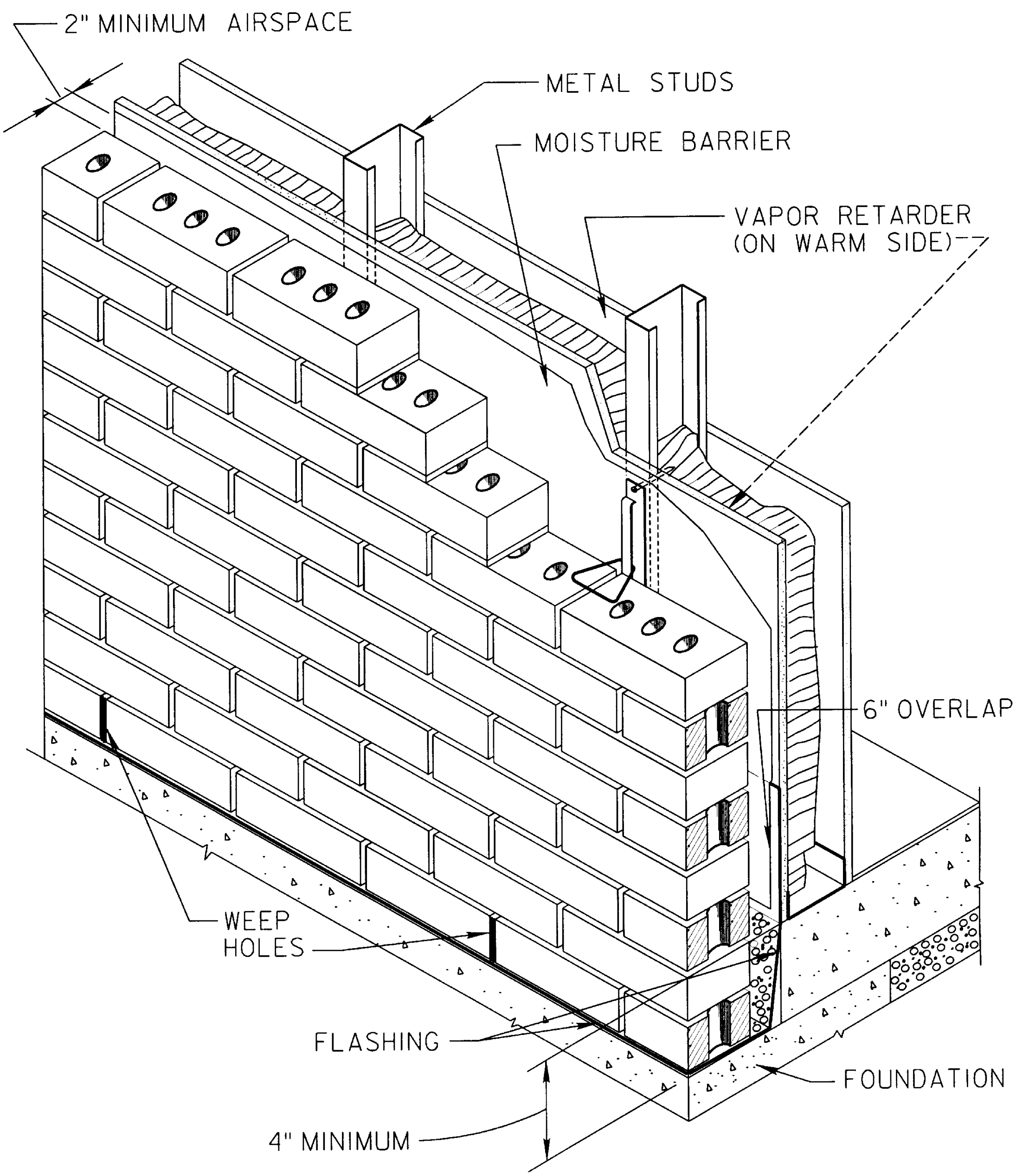
NOTE:
PLASTIC CLIP IS USED TO HOLD
1" INSULATION OVER MOISTURE
BARRIER

DRAWING NO.	SCALE: NONE	PINTLE ANCHOR
1.4	DATE: 4/17/92	

Figure G-4. Pintle Anchor

U.S. ARMY CORPS OF ENGINEERS

MASONRY VENEER/ STEEL STUD DETAIL

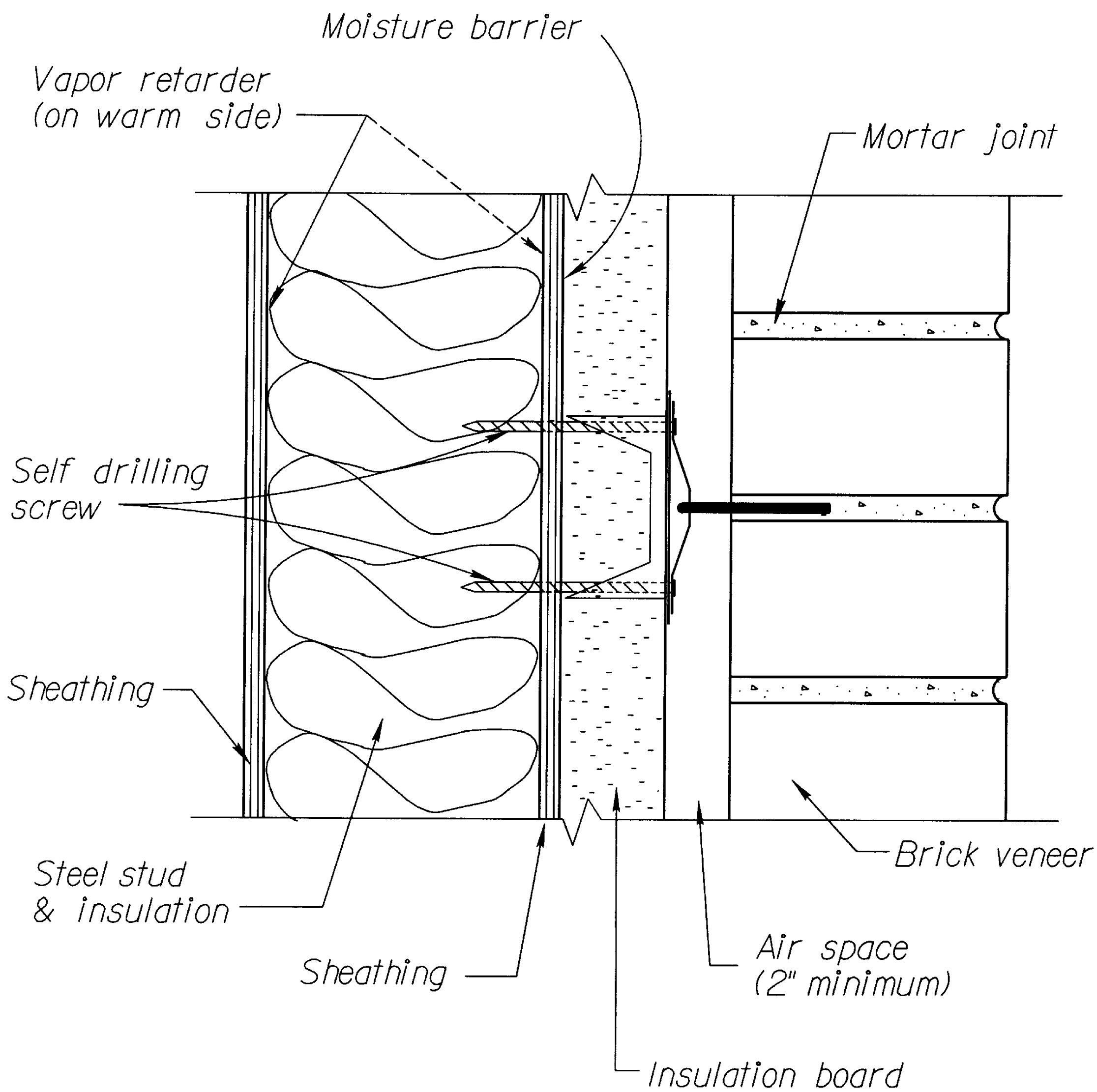


DRAWING NO.	SCALE: NONE	TYPICAL BRICK VENEER AND STEEL STUD PANEL WALL
2.1	DATE: 4/17/92	

Figure G-5. Typical Brick Veneer and Steel Stud Panel Wall

U.S. ARMY CORPS OF ENGINEERS

MASONRY VENEER/ STEEL STUD DETAIL

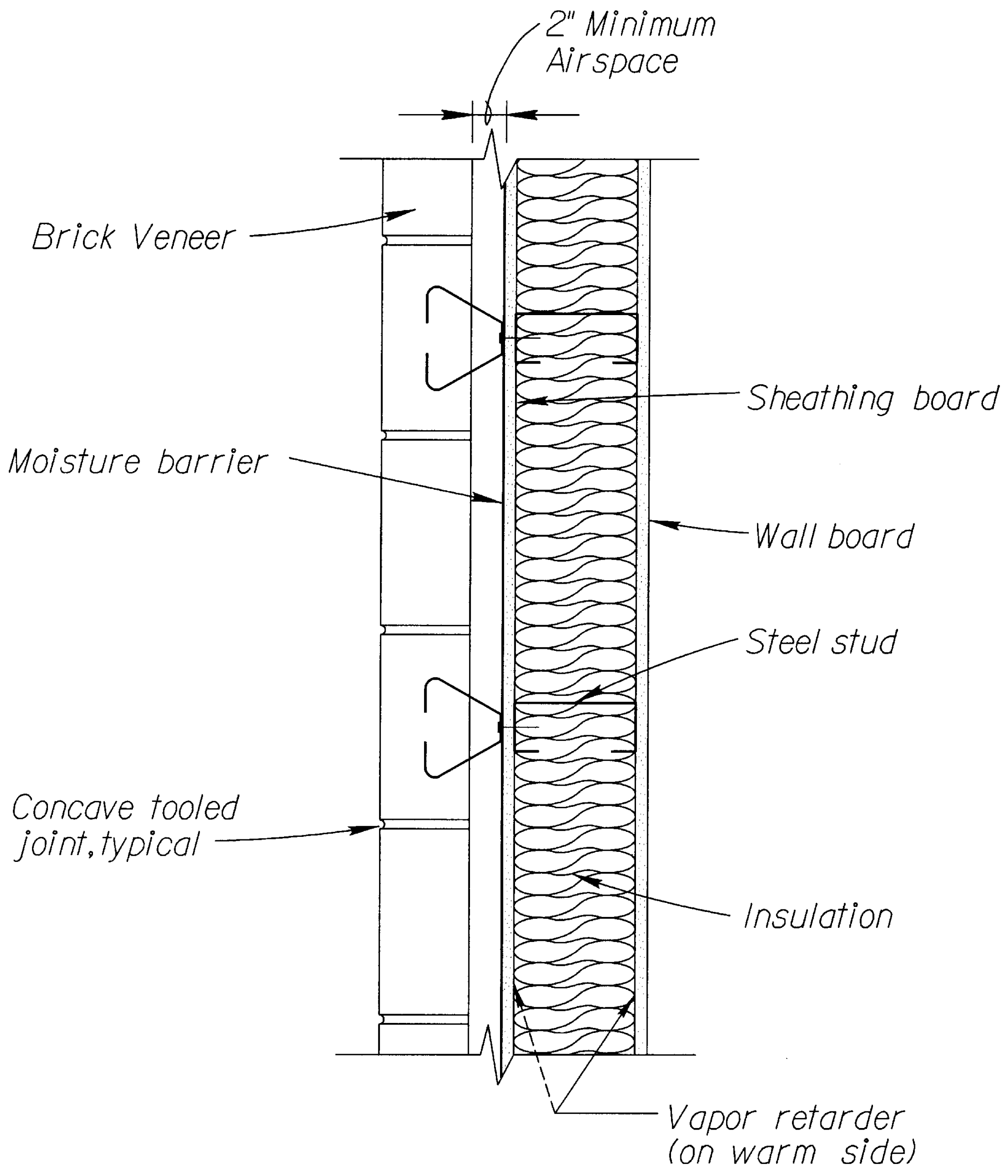


DRAWING NO.	SCALE: NONE	ADJUSTABLE WALL ANCHOR DETAIL THROUGH INSULATION BOARD
2.2	DATE: 4/17/92	

Figure G-6. Adjustable Wall Anchor Detail

U.S. ARMY CORPS OF ENGINEERS

MASONRY VENEER/ STEEL STUD DETAIL



DRAWING NO.	SCALE: 1 1/2" = 1'	ANCHORED VENEER TO STEEL STUD WALL, PLAN VIEW
2.3.1	DATE: 4/17/92	

Figure G-7. Masonry Veneer Steel Stud Panel Wall, Plan View

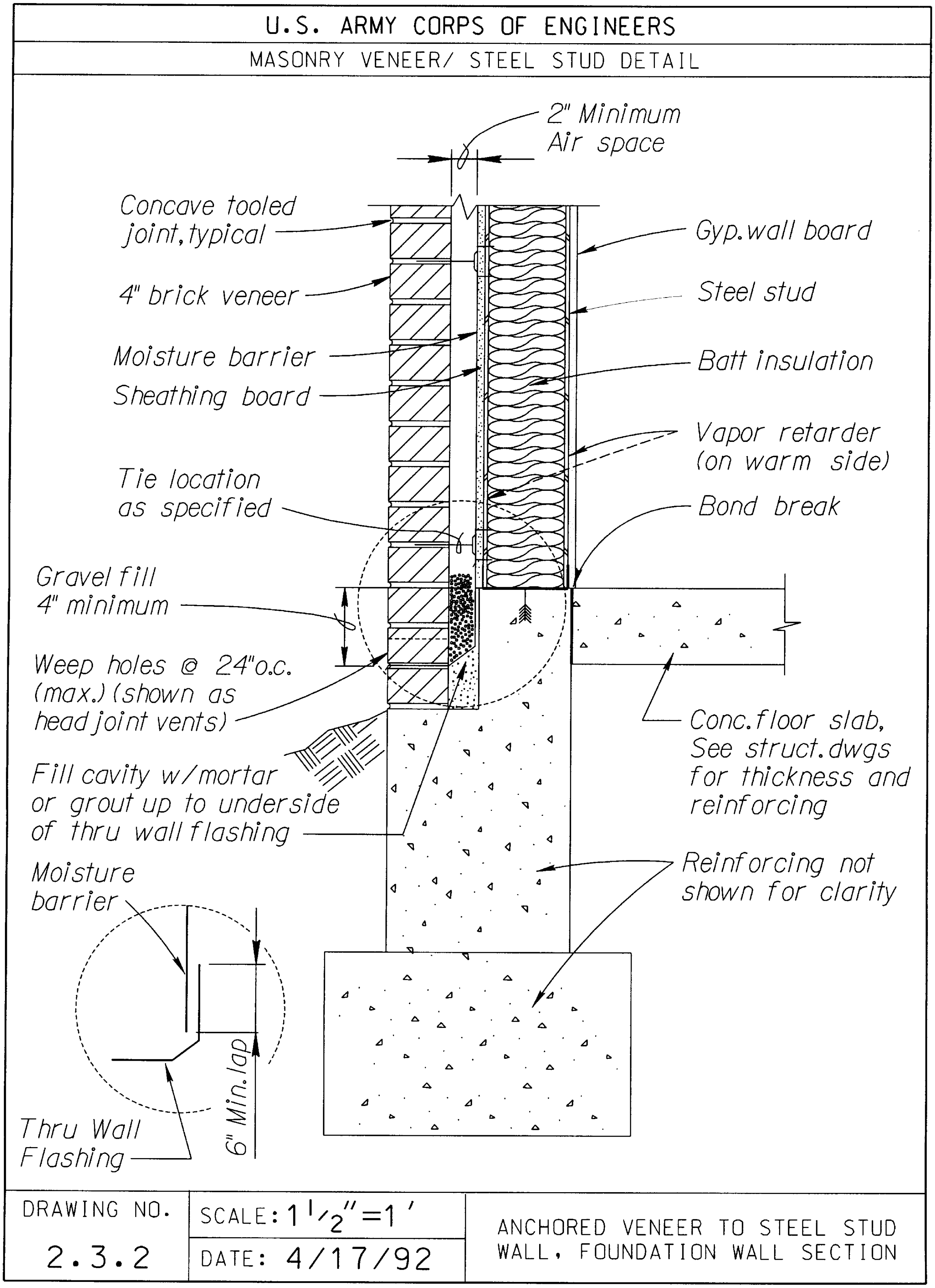


Figure G-8. Masonry Veneer Steel Stud Panel Wall, Foundation Wall Section

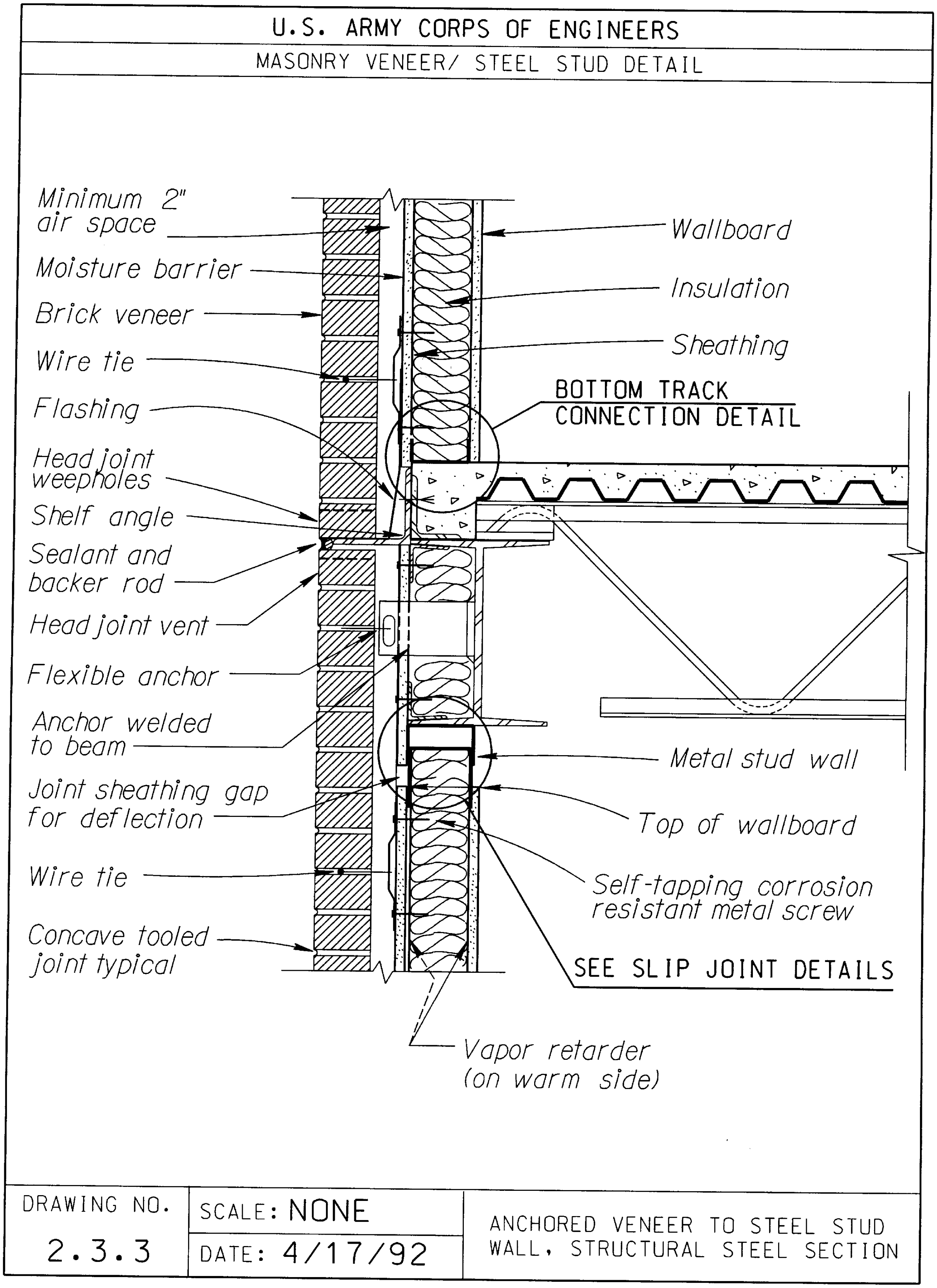


Figure G-9. Masonry Veneer Steel Stud Panel Wall, Structural Steel Section

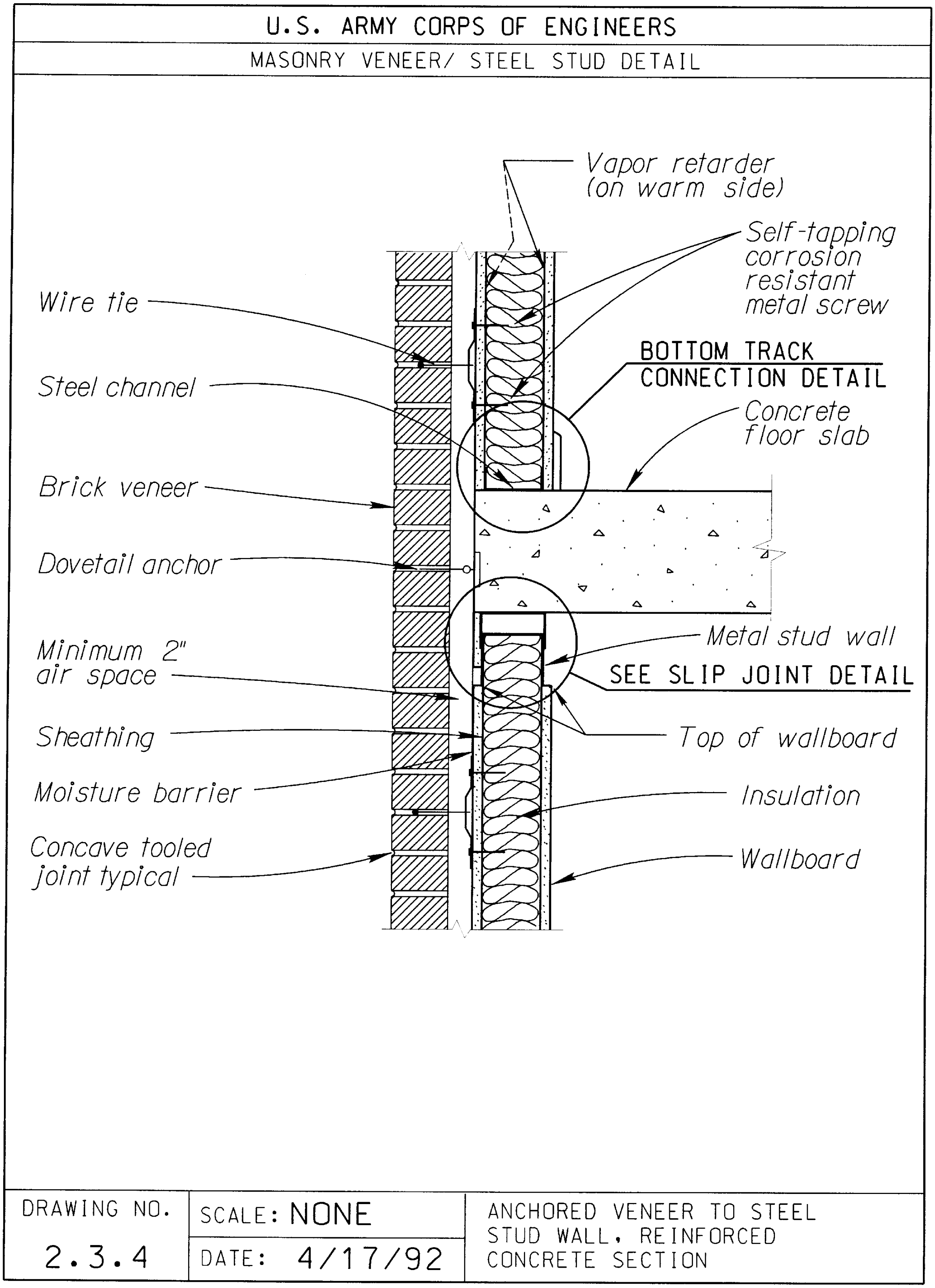
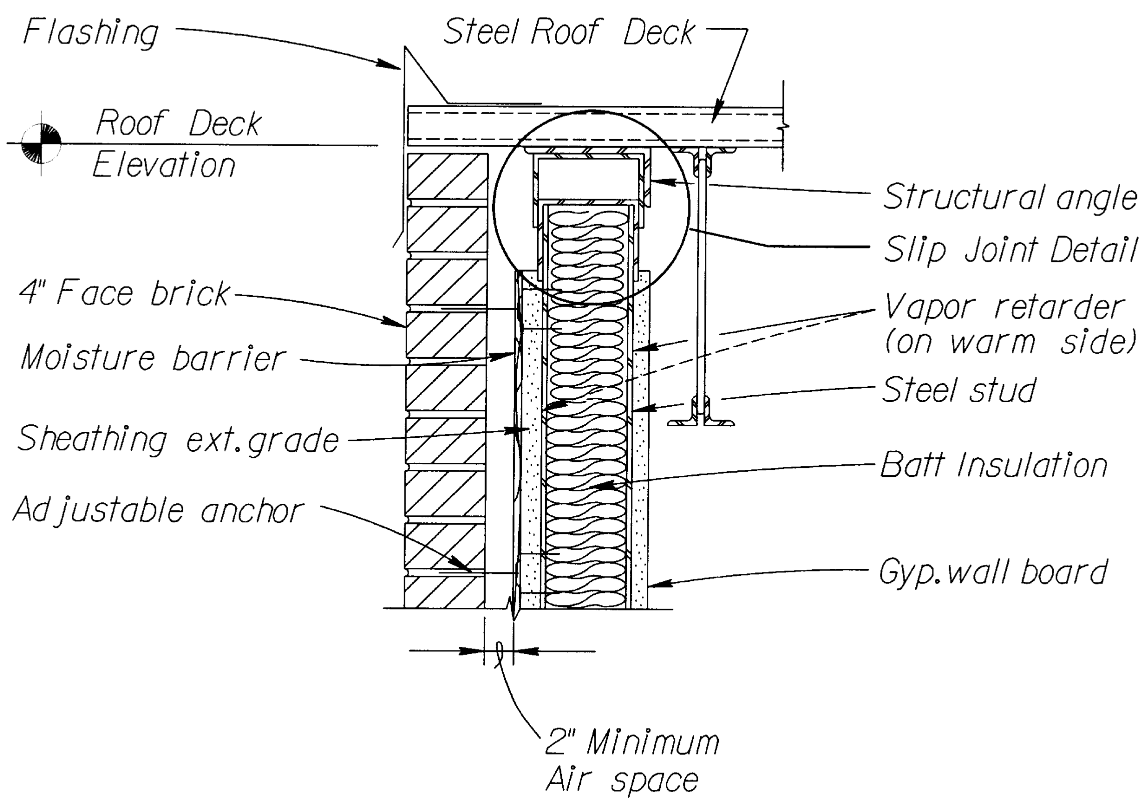


Figure G-10. Masonry Veneer Steel Stud Panel Wall, Reinforced Concrete Section

U.S. ARMY CORPS OF ENGINEERS

MASONRY VENEER/ STEEL STUD DETAIL

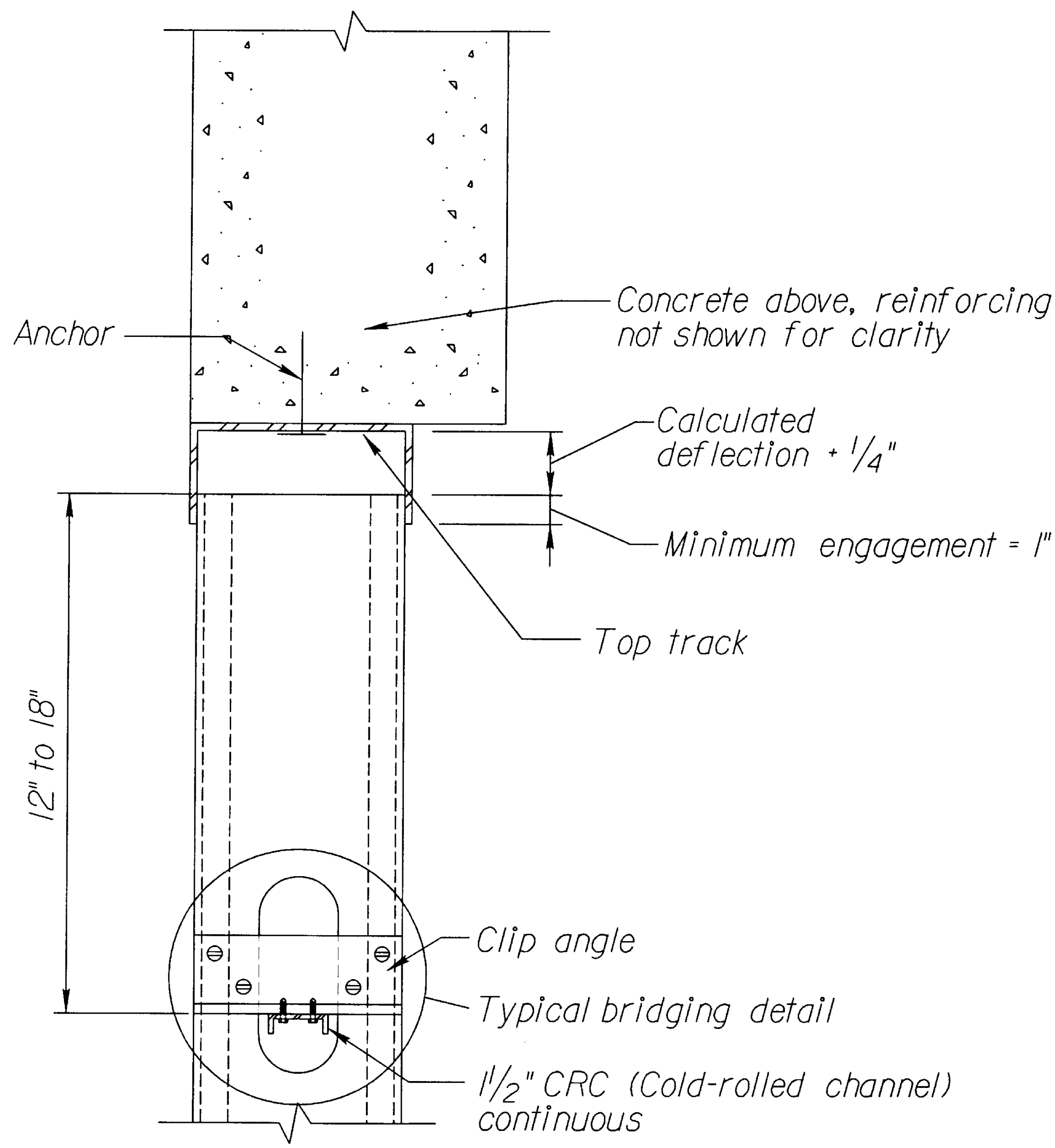


DRAWING NO.	SCALE: 1 1/2"=1'	ANCHORED VENEER TO STEEL STUD WALL, STEEL JOIST SECTION
2.3.5	DATE: 4/17/92	

Figure G-11. Masonry Veneer Steel Stud Panel Wall, Steel Joist Section

U.S. ARMY CORPS OF ENGINEERS

MASONRY VENEER/ STEEL STUD DETAIL

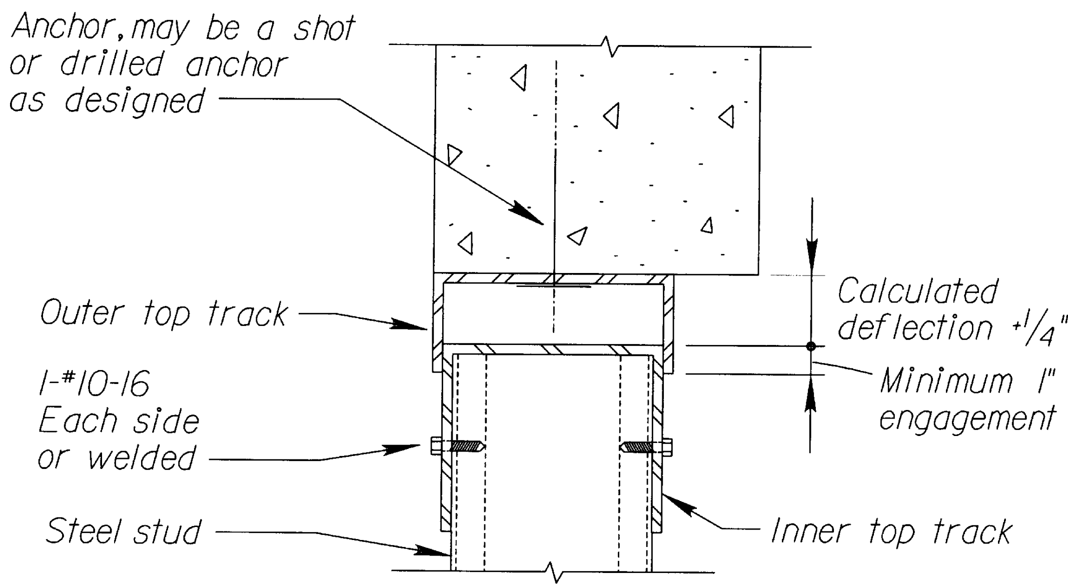


DRAWING NO.	SCALE: NONE	SINGLE TRACK SLIP JOINT DETAIL
3.1	DATE: 4/17/92	

Figure G-12. Slip Joint Details, Typical Single Track

U.S. ARMY CORPS OF ENGINEERS

MASONRY VENEER/ STEEL STUD DETAIL

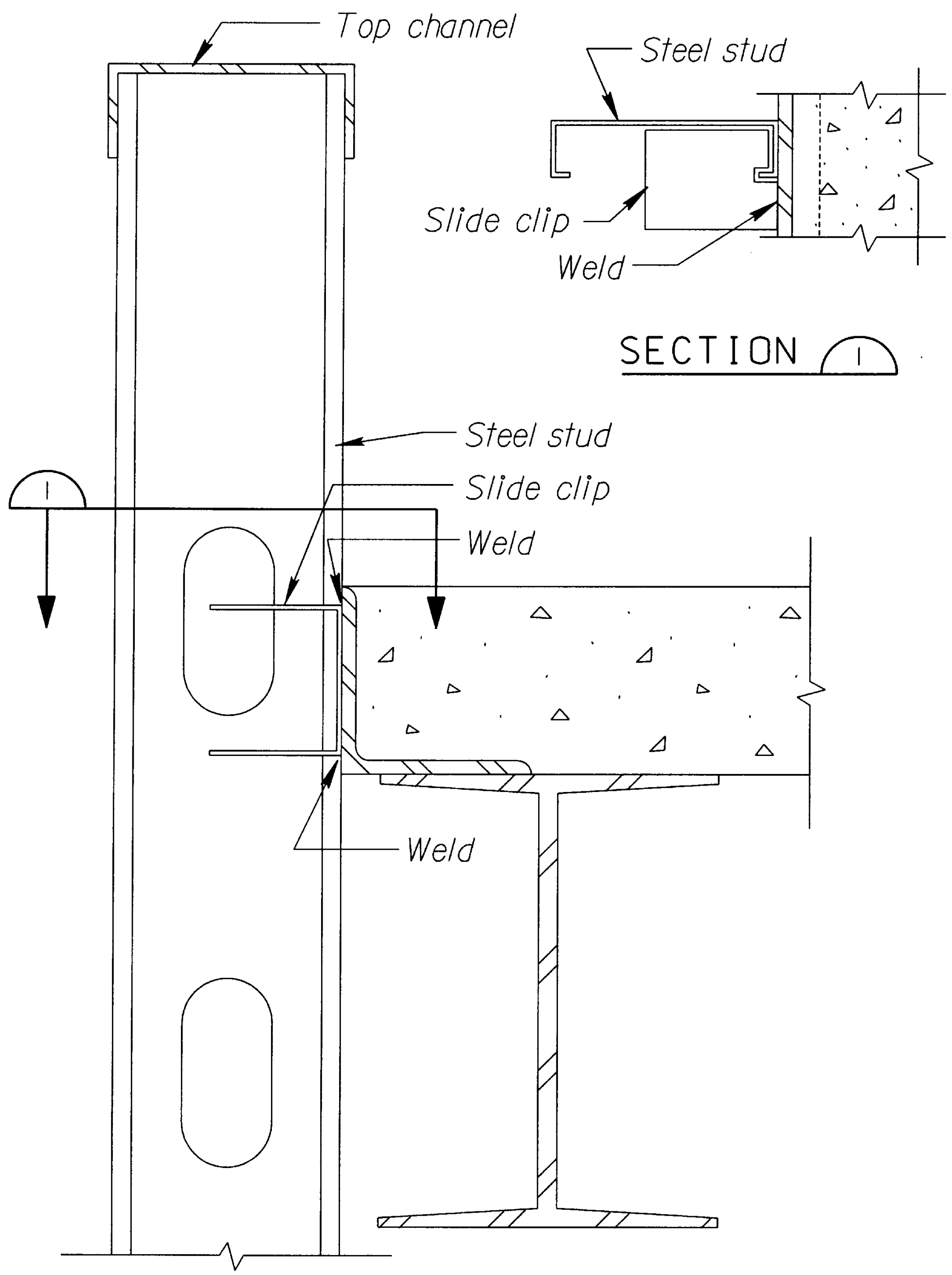


DRAWING NO.	SCALE: NONE	DOUBLE TRACK SLIP JOINT DETAIL
3.2	DATE: 4/17/92	

Figure G-13. Slip Joint Details, Typical Double Track

U.S. ARMY CORPS OF ENGINEERS

MASONRY VENEER/ STEEL STUD DETAIL

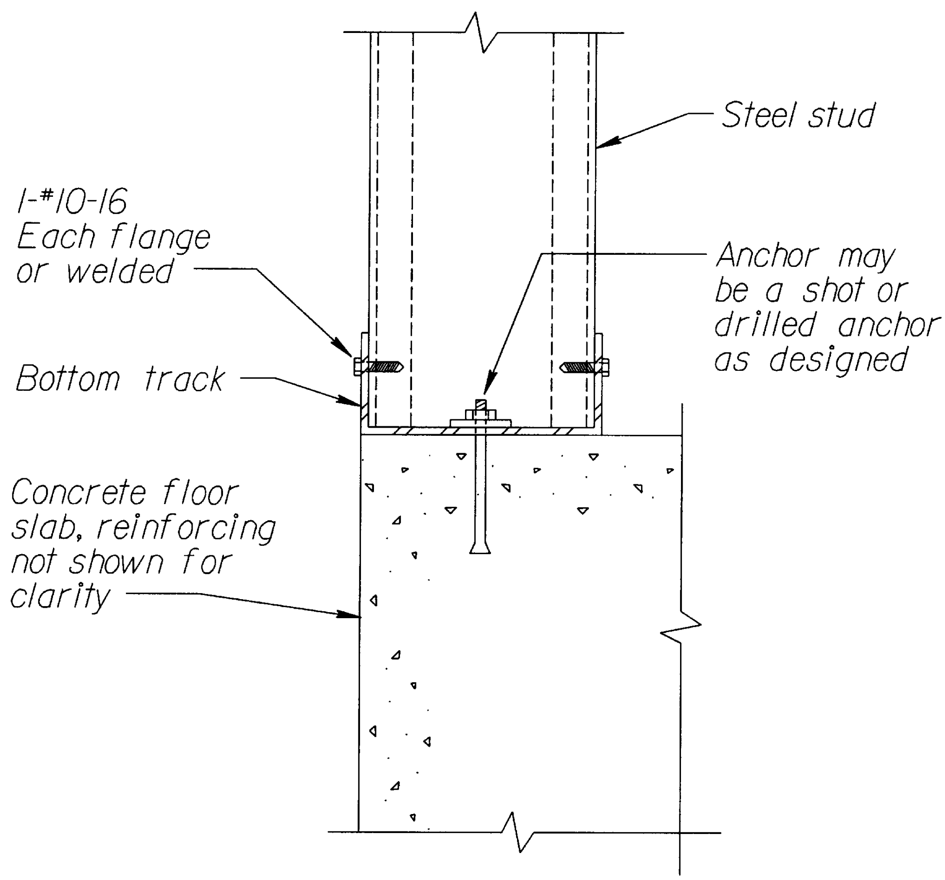


DRAWING NO. 3.3	SCALE: NONE	PARAPET SLIDE CLIP
	DATE: 4/17/92	

Figure G-14. Slip Joint Details, Parapet Slide Clip

U.S. ARMY CORPS OF ENGINEERS

MASONRY VENEER/ STEEL STUD DETAIL



DRAWING NO.	SCALE: NONE	COLD-FORMED BOTTOM CONNECTION, TRACK ANCHORED TO CONCRETE
4.1	DATE: 4/17/92	

Figure G-15. Bottom Connection, Track Anchored to Concrete

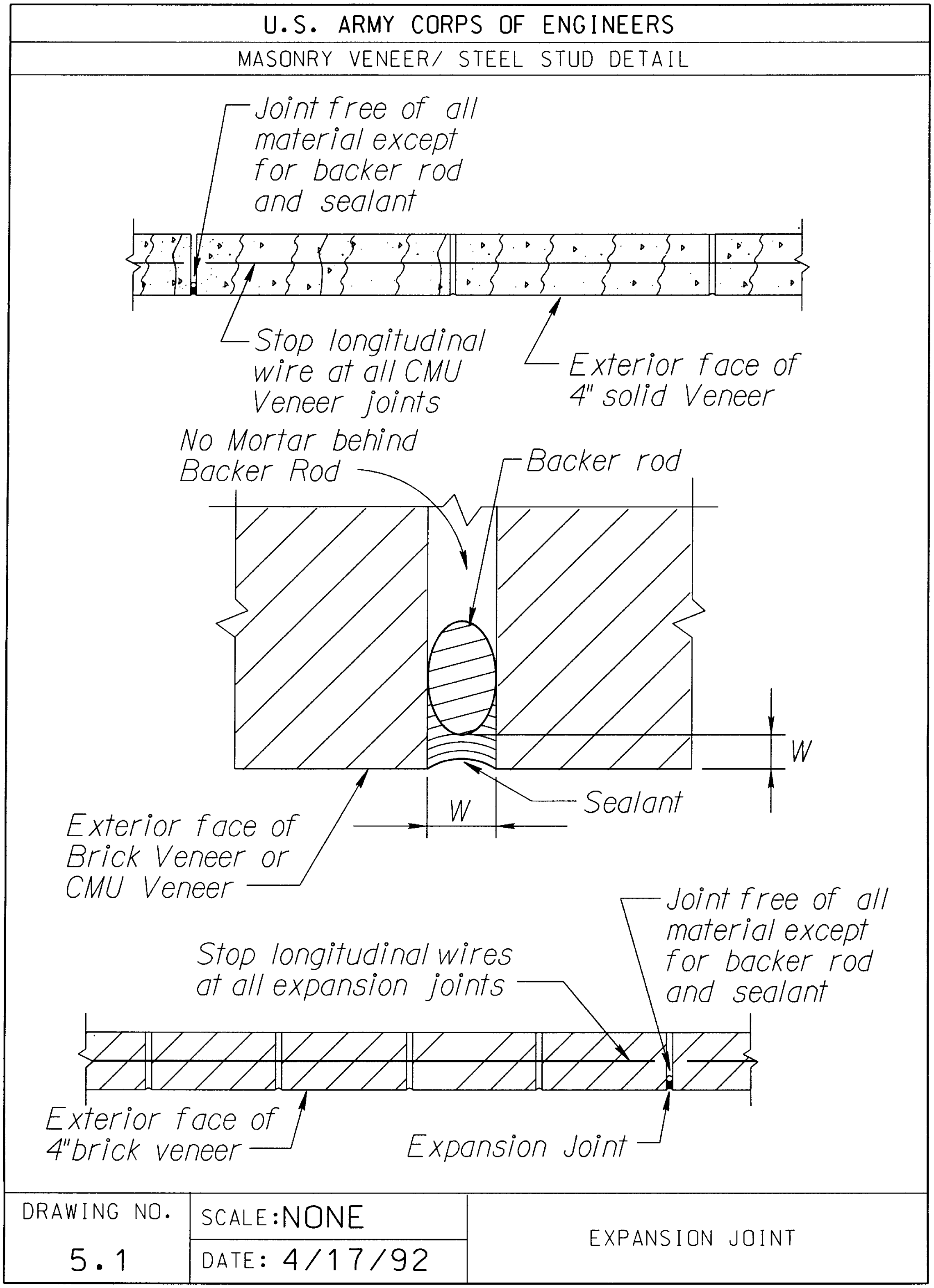


Figure G-16. Expansion Joints, Brick or CMU Veneer Joint

APPENDIX H

Metric Conversion Data Sheet

Quantity	*Unit*	*Symbol*
length	meter	m
mass	kilogram	kg
time	second	s
force	newton	N= kg m/s^2
stress	pascal	Pa= N/m^2
energy	joule	J= N-m

Prefix	*Symbol*	*Order of Magnitude*
mega	M	10^6
kilo	k	10^3
milli	m	10^{-3}

METRIC CONVERSIONS

Multiply	*by*	*to obtain*
AREA AND VOLUME		
ft^2	0.092 903	m^2
ft^3	0.028 136 847	m^3
gal	0.003 785 412	m^3
in^2	645.160 000	mm^2
SECTION MODULUS		
in^3	16 387.064	mm^3
MOMENT OF INERTIA		
in^4	416 231.430	mm^4
LENGTH		
foot (ft)	304.800	millimeters (mm)
inch (in)	25.400	millimeters (mm)
MASS		
kg (force/mass)	9.806 650	N
pound-mass (lbm)	0.453 592	kilogram (kg, mass)
FORCE		
pound-force (lbf)	4.448 222	N
plf	14.593 903	N/m
TORQUE		
lb-in	0.112 985	N-m
lb-ft	1.355 818	N-m

Multiply	*by*	*to obtain*
PRESSURE AND DENSITY		
foot of water	2.988 980	kPa
g/cm^3	62.427 900	pcf
g/cm^3	9.806 650	kN/m^3
kg/cm^2	98.066 500	kPa
ksi	6.894 757	MPa, N/mm^2
pcf	16.018 460	kg/m^3
pcf	157.087 616	N/m^3
psf	47.880 260	Pa
psi	6.894 757	kPa

APPENDIX I

STANDARD DRAWINGS COLD-FORMED STEEL

This Appendix links designers to a standard CADD library of Cold-Formed Steel details in Intergraph format that are for general information only. These details are available to designers for use in Military design projects. They were developed originally by AISI for residential construction and should be modified according for larger projects. They include typical; floor, roof, and wall framing, plans and elevations. Also included are typical roof truss elevations, and connection details for floors, walls, openings, and roofs. Connection details can be fastened using bolts welds, or screws. Punch-outs are not shown but are acceptable and vary in size and configuration based on the manufacturer. Additional design and detailing is required before this information can be used in construction documents. Neither the Corps of Engineers nor AISI is responsible for the proper use of this information. Before specifying or using cold-formed material a competent Structural engineer shall design and check the adequacy of the design and any cold-formed component used in the design. Anyone using this information assumes all liability arising from such use.

Details for moisture protection, thermal insulation, seismic conditions, and high wind conditions are special requirements that need to be considered in any design using cold-formed steel. Designers shall use the information from Chapter 4 and Appendix G when designing for moisture protection and thermal insulation of steel stud systems.

Standard Detail Drawings for Cold-formed Steel Systems:

Stls 101.pdf: Schematic

Stls 102.pdf: Exterior Walls

Stls 103.pdf: Exterior Walls

Stls 104.pdf: Interior Framing Detail

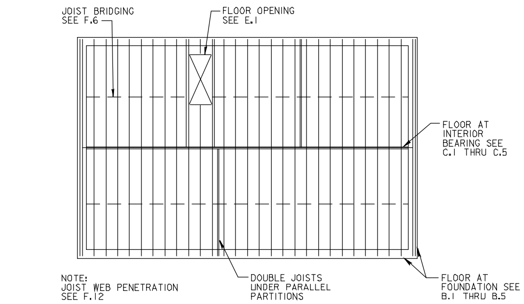
Stls 105.pdf: Roof Member Connections

Stls 106.pdf: Framing Details for Floor and Wall Openings

Stls 107.pdf: Special Reinforcement Bridging, Blocking, & Miscellaneous

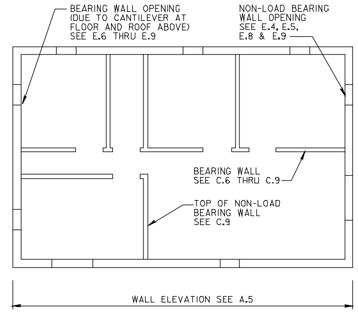
Stls 108.pdf: Special Reinforcement Bridging, Blocking, & Miscellaneous

Stls 109.pdf: Special Reinforcement Bridging, Blocking, & Miscellaneous

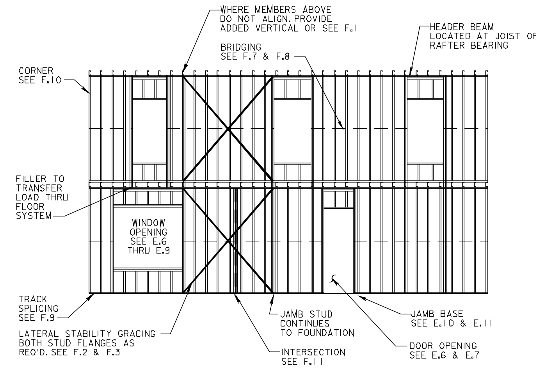


NOTE:
JOIST WEB PENETRATION
SEE F.12

LOWER FLOOR FRAMING PLAN A.1 SXISY
NO SCALE

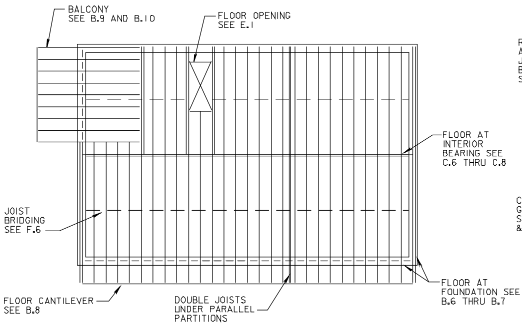


WALL LAYOUT PLAN A.3 SXISY
NO SCALE

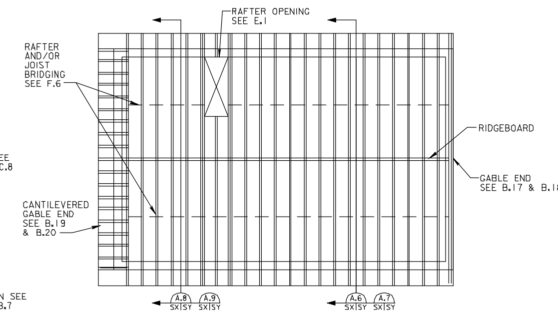


WALL FRAMING ELEVATION A.5 SXISY
NO SCALE

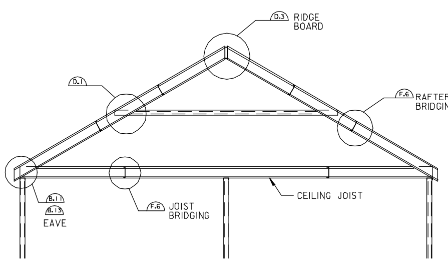
NOTES:
1. JOISTS ALIGN OVER WALL STUDS (TYP.)
2. JAMB MEMBERS MUST BE CARRIED DOWN ALL WALLS TO FOUNDATION (TYP.).
3. STUD WEB PENETRATIONS, SEE F.12.
4. HEADERS FOR OPENINGS MAY BE LOCATED DIRECTLY ABOVE OPENING OR AT JOIST BEARING. WHEN LOCATED AT WINDOW HEAD, CRIPPLE STUDS MUST BE TIGHTLY SEATED FOR FULL BEARING.



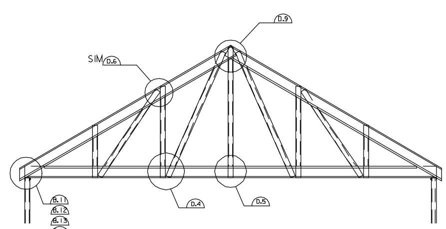
UPPER FLOOR FRAMING PLAN A.2 SXISY
NO SCALE



ROOF FRAMING PLAN A.4 SXISY
NO SCALE

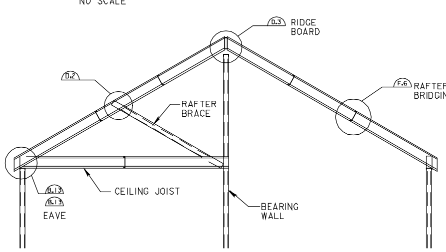


ROOF COLLAR TIE FRAMING ELEVATION A.6 SXISY
NO SCALE

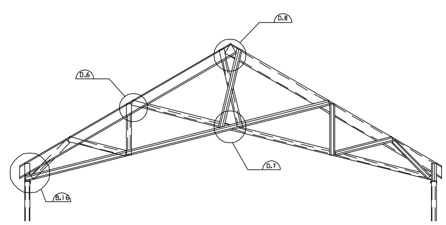


ROOF TRUSS ELEVATION A.8 SXISY
NO SCALE

NOTE:
MODIFY QUANTITY OF
WEB MEMBERS AS REQ'D.
BASED ON SPAN.



ROOF RAFTER FRAMING ELEVATION A.7 SXISY
NO SCALE



ROOF SCISSORS TRUSS ELEVATION A.9 SXISY
NO SCALE

NOTE:
MODIFY QUANTITY OF
WEB MEMBERS AS REQ'D.
BASED ON SPAN.

FOR REVIEW ONLY

$$ – THINK VALUE ENGINEERING – $$

Revisions

Symbol	Descriptions	Date	Approved

US ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
OMAHA, NE

Designed by: X
Drawn by: X
Checked by: X
Reviewed by:
Submitted by:
Chief: [redacted] Section

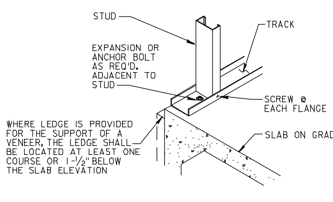
AIR FORCE STANDARDS
PRESSURIZED HYDRANT FUELING SYSTEM
TYPE III

SCHEMATIC PLANS

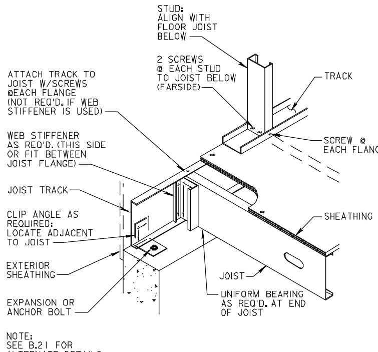
Plot Scale Ratio: 4:1
Design File: STD01/OCE/STLS101.DGN
Date:
Spec. No.:
Drawing Code:
Sheet reference number: S-1

COMPUTER AIDED DESIGN & DRAFTING

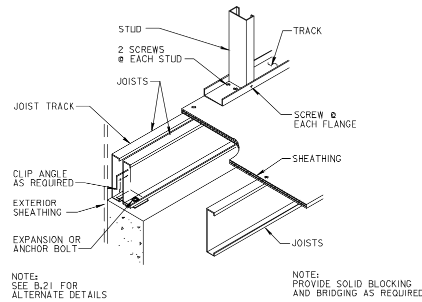
D



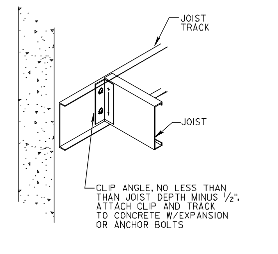
BASE OF WALL AT SLAB ON GRADE (B.1 / SX1SY)
NO SCALE



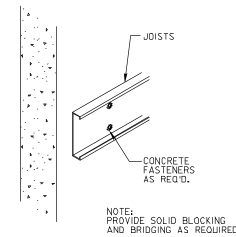
FLOOR JOISTS BEARING ON FOUNDATION (B.2 / SX1SY)
NO SCALE



FLOOR JOISTS PARALLEL TO FOUNDATION (B.3 / SX1SY)
NO SCALE

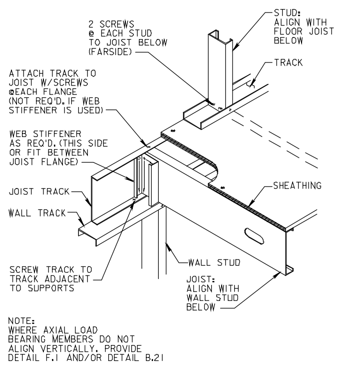


FLOOR JOIST SUPPORT AT CONTINUOUS WALL (B.4 / SX1SY)
NO SCALE

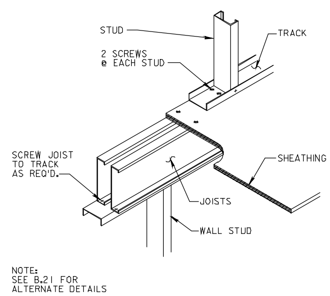


FLOOR JOIST PARALLEL TO CONTINUOUS WALL (B.5 / SX1SY)
NO SCALE

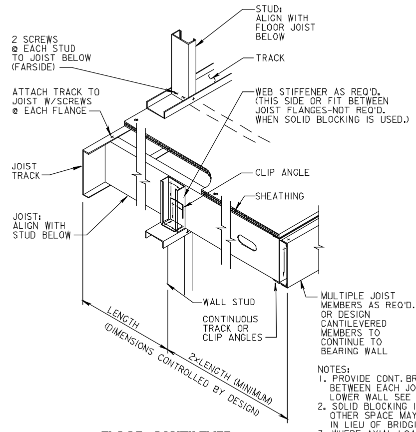
C



FLOOR FRAMING AT EXTERIOR WALL (B.6 / SX1SY)
NO SCALE

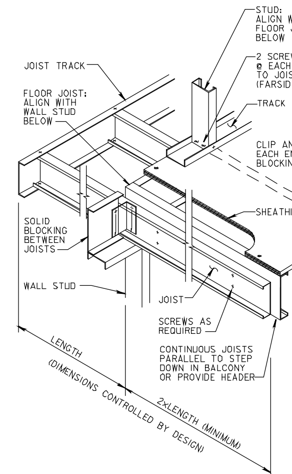


FLOOR JOISTS PARALLEL TO EXTERIOR WALL (B.7 / SX1SY)
NO SCALE



NOTES:
1. PROVIDE CONT. BRIDGING BETWEEN EACH JOIST AT LOWER WALL SEE F.6.
2. SOLID BLOCKING IN EVERY OTHER SPACE MAY BE USED IN LIEU OF BRIDGING-SEE F.4
3. WHERE AXIAL LOAD BEARING MEMBERS DO NOT ALIGN VERTICALLY PROVIDE DETAIL F.1.

FLOOR CANTILEVER (B.8 / SX1SY)
NO SCALE

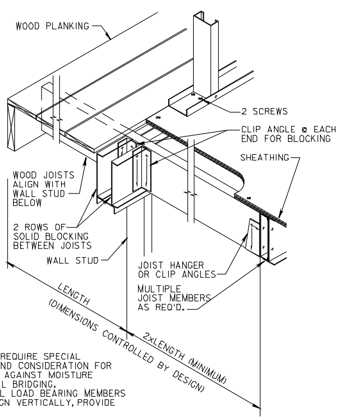


NOTES:
1. BALCONIES REQUIRE SPECIAL DETAILING AND CONSIDERATION FOR PROTECTION AGAINST MOISTURE AND THERMAL BRIDGING.
2. WHERE AXIAL LOAD BEARING MEMBERS DO NOT ALIGN VERTICALLY, PROVIDE DETAIL F.1

BALCONY WITH STEP DOWN (B.9 / SX1SY)
NO SCALE

B

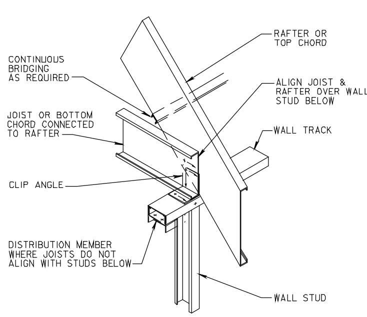
FOR REVIEW ONLY



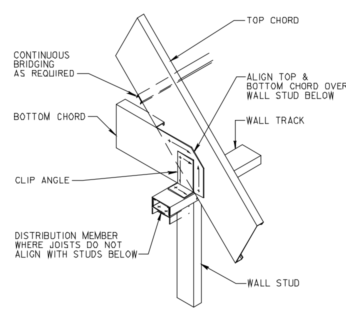
NOTES:
1. BALCONIES REQUIRE SPECIAL DETAILING AND CONSIDERATION FOR PROTECTION AGAINST MOISTURE AND THERMAL BRIDGING.
2. WHERE AXIAL LOAD BEARING MEMBERS DO NOT ALIGN VERTICALLY, PROVIDE DETAIL F.1

WOOD DECK BALCONY (B.10 / SX1SY)
NO SCALE

A



ROOF EAVE (B.11 / SX1SY)
NO SCALE



ROOF TRUSS EAVE (B.12 / SX1SY)
NO SCALE

$$ – THINK VALUE ENGINEERING – $$

Revisions

Symbol	Descriptions	Date	Approved

US ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
OMAHA, NE

Designed by: X
Drawn by: X
Checked by: X
Reviewed by:
Submitted by:

AIR FORCE STANDARDS
PRESSURIZED HYDRANT FUELING SYSTEM
TYPE III

DETAILS AT EXTERIOR WALLS

Plot Scale Ratio: 4:1
Date:
Design File: STD01OCE5/STLS102.DGN
Spec. No.:
Drawing Code:
Sheet reference number: S-2

COMPUTER AIDED DESIGN & DRAFTING

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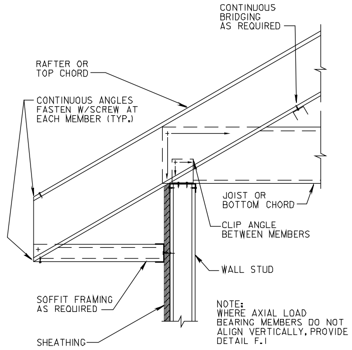
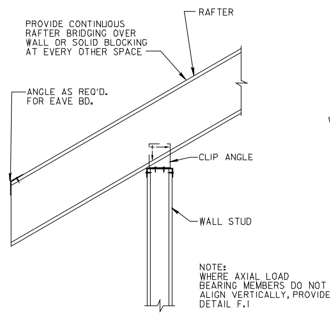
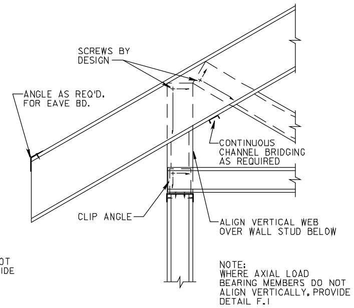


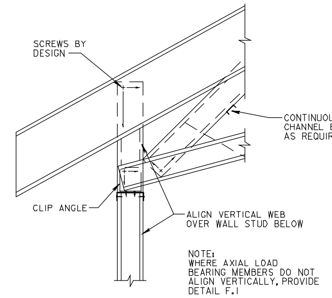
ROOF EAVE AND SOFFIT (6.13 / SXISY)
NO SCALE



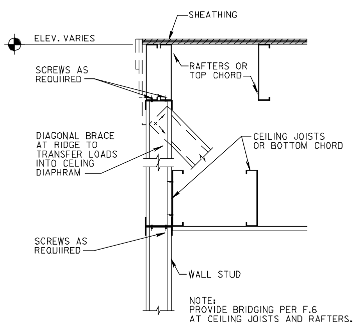
ROOF EAVE AND CATHEDRAL CEILING (6.14 / SXISY)
NO SCALE



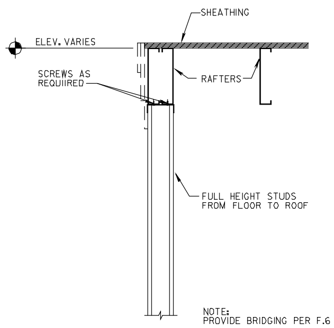
ROOF TRUSS BEARING (6.15 / SXISY)
NO SCALE



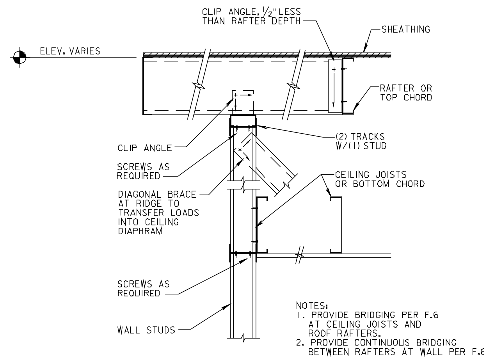
ROOF SCISSORS TRUSS BEARING (6.16 / SXISY)
NO SCALE



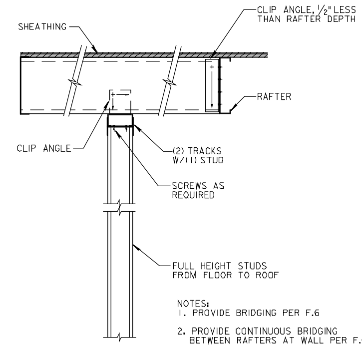
ROOF GABLE END (6.17 / SXISY)
NO SCALE



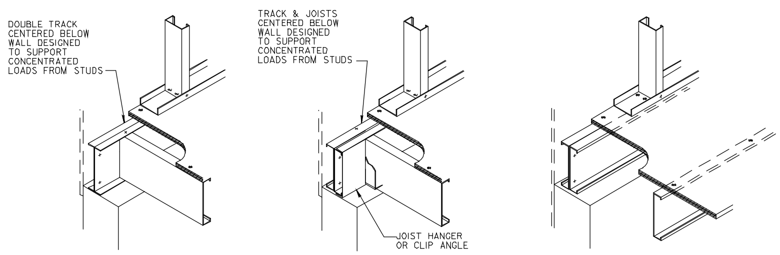
ROOF GABLE END AT CATHEDAL (6.18 / SXISY)
NO SCALE



CANTILEVERED ROOF CABLE END (6.19 / SXISY)
NO SCALE



CANTILEVERED GABLE END AT CATHEDRAL (6.20 / SXISY)
NO SCALE



DOUBLE TRACK DETAIL

JOISTS PERPENDICULAR TO WALL

JOISTS PARALLEL TO WALL

ALTERNATE RIBBON JOIST DETAILS (6.21 / SXISY)
NO SCALE

FOR REVIEW ONLY

$$ – THINK VALUE ENGINEERING – $$

Revisions

Symbol	Descriptions	Date	Approved

US ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
OMAHA, NE

Designed by: X
Drawn by: X
Checked by: X
Reviewed by:
Submitted by:

AIR FORCE STANDARDS
PRESSURIZED HYDRANT FUELING SYSTEM
TYPE III

DETAILS AT EXTERIOR WALLS

Plot Scale Ratio: 4:1		Date:	Sheet reference number:
Design File: STD01OCE/STLS103.DGN			
Spec. No.:		Drawing Code:	S-3

COMPUTER AIDED DESIGN & DRAFTING

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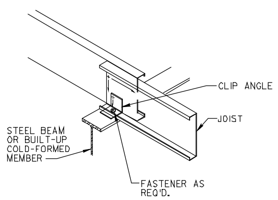
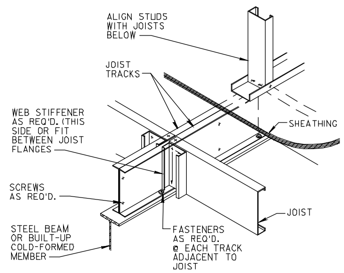
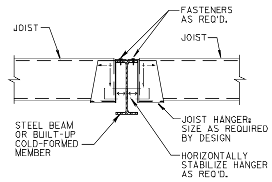


NOTE:
1. CONTINUOUS BRIDGING REQ'D. BETWEEN EACH JOIST ABOVE BEAM-SEE F.6. SOLID BLOCKING IN EVERY OTHER SPACE MAY BE USED IN LIEU OF BRIDGING.
2. WHEN WALL ABOVE, STUDS MUST ALIGN WITH JOISTS.

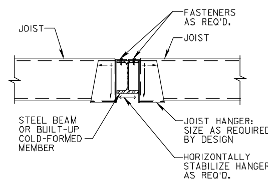
FLOOR JOISTS SUPPORTED BY BEAM (OVERLAPPED) C.1 SXISY
NO SCALE



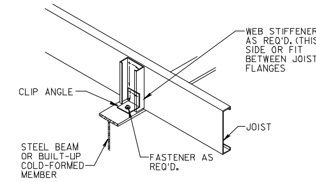
JOIST SUPPORTED BY BEAM (ALTERNATE) C.2 SXISY
NO SCALE



JOIST SUPPORTED BY STEEL BEAM (UPSET DEEP BEAM) C.3 SXISY
NO SCALE

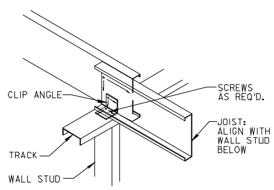


JOIST SUPPORTED BY STEEL BEAM (UPSET SHALLOW BEAM) C.4 SXISY
NO SCALE



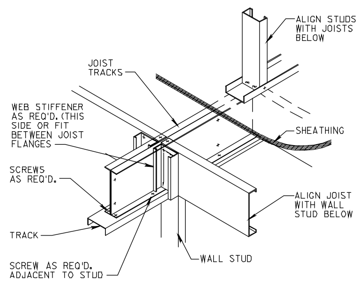
NOTE:
1. CONTINUOUS BRIDGING REQ'D. BETWEEN EACH JOIST ABOVE BEAM-SEE F.6. SOLID BLOCKING IN EVERY OTHER SPACE MAY BE USED IN LIEU OF BRIDGING.
2. WHEN WALL ABOVE, STUDS MUST ALIGN WITH JOISTS.
3. WEB STIFFENERS ARE NOT REQUIRED WHEN CONTINUOUS SOLID BLOCKING IS USED.

JOISTS OVER BEAM (CONTINUOUS SPAN) C.5 SXISY
NO SCALE



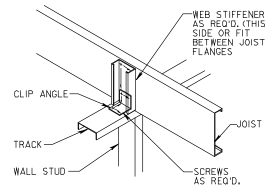
NOTE:
1. CONTINUOUS BRIDGING REQ'D. BETWEEN EACH JOIST ABOVE BEAM-SEE F.6. SOLID BLOCKING IN EVERY OTHER SPACE MAY BE USED IN LIEU OF BRIDGING.
2. WHEN WALL ABOVE, STUDS MUST ALIGN WITH JOISTS.
3. WHERE AXIAL LOAD BEARING MEMBERS DO NOT ALIGN VERTICALLY, PROVIDE DETAIL F.1

JOIST SUPPORTED ON STUDS C.6 SXISY
NO SCALE



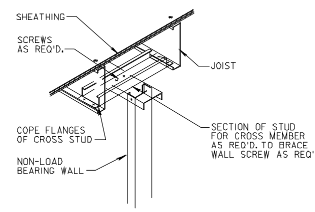
NOTES:
1. WHEN JOIST TRACK IS DESIGNED TO SUPPORT JOISTS AND/OR WALL ABOVE BETWEEN STUDS.
2. WHERE AXIAL LOAD BEARING MEMBERS DO NOT ALIGN VERTICALLY, PROVIDE DETAIL F.1

JOIST SUPPORTED ON STUDS (ALTERNATE) C.7 SXISY
NO SCALE

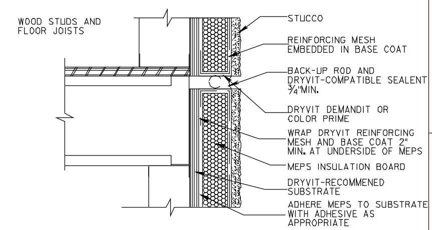


NOTE:
1. CONTINUOUS BRIDGING REQ'D. BETWEEN EACH JOIST ABOVE BEAM-SEE F.6. SOLID BLOCKING IN EVERY OTHER SPACE MAY BE USED IN LIEU OF BRIDGING.
2. WHEN WALL ABOVE, STUDS MUST ALIGN WITH JOISTS.
3. WEB STIFFENERS ARE NOT REQUIRED WHEN CONTINUOUS SOLID BLOCKING IS USED.
4. WHERE AXIAL LOAD BEARING MEMBERS DO NOT ALIGN VERTICALLY, PROVIDE DETAIL F.1

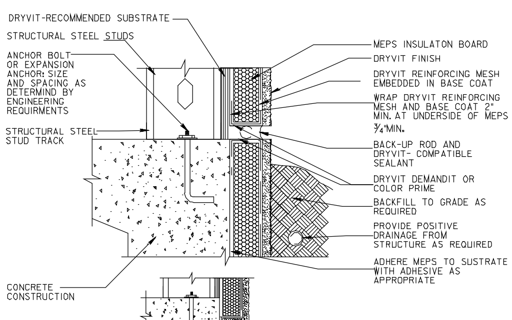
JOISTS SUPPORTED ON STUDS (CONTINUOUS SPAN) C.8 SXISY
NO SCALE



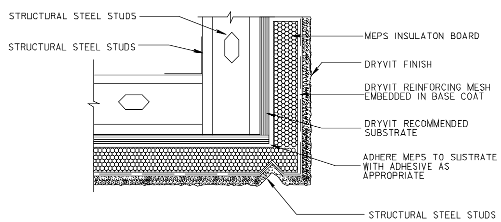
TOP OF NON-LOAD BEARING WALL PARALLEL TO JOISTS C.9 SXISY
NO SCALE



INTERMEDIATE LEVEL/WOOD FRAMING EXPANSION JOINT AT FLOOR LEVEL C.9 SXISY
NO SCALE



GRADE LEVEL/LIGHT GAUGE EXPANSION JOINT FRAMING AT CONCRETE SLAB C.9 SXISY
NO SCALE



FOR REVIEW ONLY

$$ – THINK VALUE ENGINEERING – $$

Revisions

Symbol	Description	Date	Approved

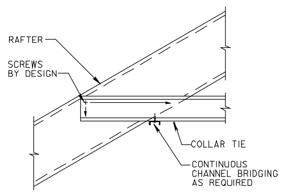
US ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
OMAHA, NE

Designed by: X
Drawn by: X
Checked by: X
Reviewed by:
Submitted by:

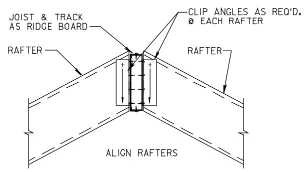
AIR FORCE STANDARDS
PRESSURIZED HYDRANT FUELING SYSTEM
TYPE III
INTERIOR FRAMING DETAILS

Plot Scale Ratio: 4:1
Design File: STD01OCE5/STLS104.DGN
Spec. No.:
Date:
Drawing Code:
Sheet reference number: S-4

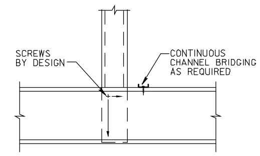
COMPUTER AIDED DESIGN & DRAFTING



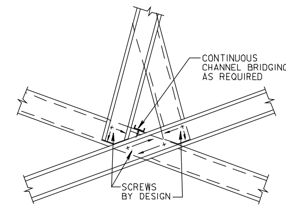
COLLAR TIE AT RAFTER D.1 SX1SY
NO SCALE



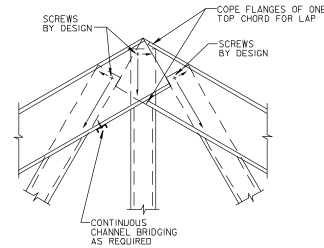
RIDGE BOARD D.3 SX1SY
NO SCALE



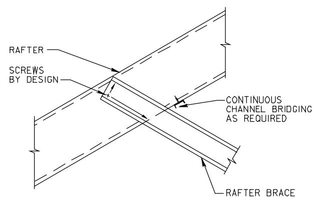
WEB TO BOTTOM CHORD OF TRUSS D.5 SX1SY
NO SCALE



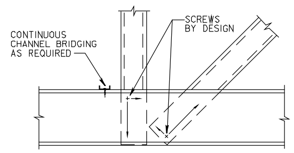
BOTTOM CHORD INTERSECTION ON SCISSORS TRUSS D.7 SX1SY
NO SCALE



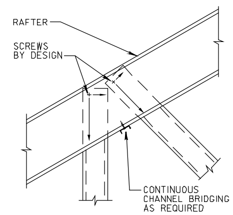
WEBS AT RIDGE OF TRUSS D.9 SX1SY
NO SCALE



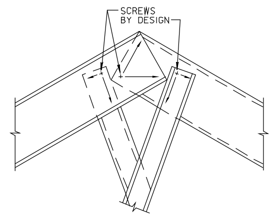
RAFTER BRACE D.2 SX1SY
NO SCALE



WEBS TO BOTTOM CHORD OF TRUSS D.4 SX1SY
NO SCALE



WEBS TO TOP CHORD OF TRUSS D.6 SX1SY
NO SCALE



WEBS AT RIDGE OF SCISSORS TRUSS D.8 SX1SY
NO SCALE

FOR REVIEW ONLY

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Revisions

Symbol	Descriptions	Date	Approved

US ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
OMAHA, NE

Designed by: X
Drawn by: X
Checked by: X
Reviewed by:
Submitted by:

AIR FORCE STANDARDS
PRESSURIZED HYDRANT FUELING SYSTEM
TYPE III

ROOF MEMBER CONNECTIONS

Plot Scale Ratio: 1:1
Date:
Design File: STD01OCE5STLS105.DGN
Spec. No.:
Drawing Code:
Sheet reference number: S-5

COMPUTER AIDED DESIGN & DRAFTING

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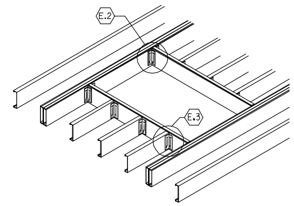
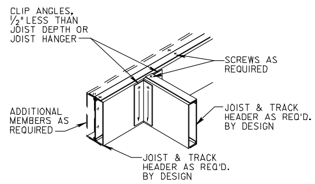
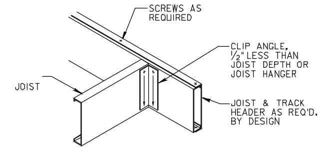


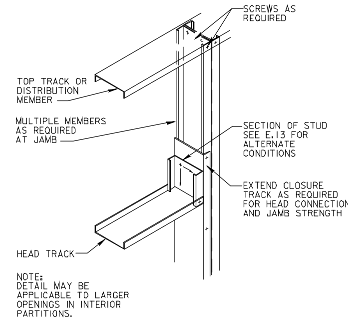
OPENING IN JOISTS/RAFTERS (E.1 SXTSY)
NO SCALE



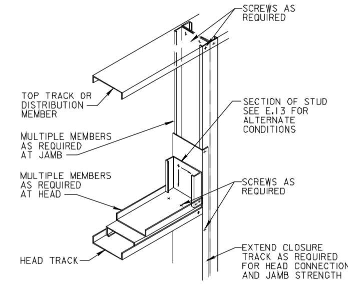
JOIST HEADER TO BUILT-UP JOISTS (E.2 SXTSY)
NO SCALE



JOIST TO JOIST HEADER (E.3 SXTSY)
NO SCALE

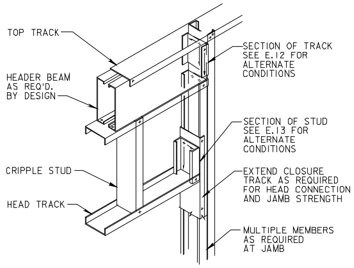


NON-AXIAL LOAD BEARING JAMB & HEAD
[<4'-0" (1.2m) OPENING WIDTH] (E.4 SXTSY)
NO SCALE

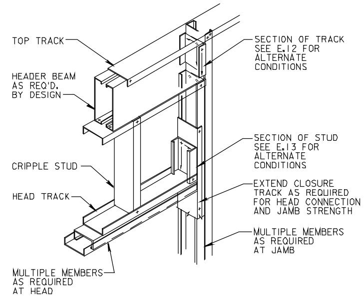


NON-AXIAL LOAD BEARING JAMB & HEAD
[>4'-0" (1.2m) OPENING WIDTH] (E.5 SXTSY)
NO SCALE

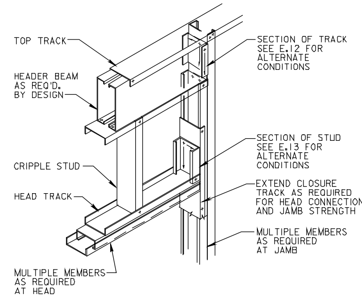
C



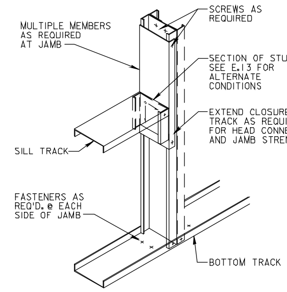
AXIAL LOAD BEARING JAMB & HEAD
[<4'-0" (1.2m) OPENING WIDTH] (E.6 SXTSY)
NO SCALE



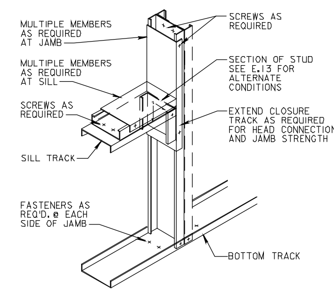
AXIAL LOAD BEARING JAMB & HEAD
[>4'-0" (1.2m) OPENING WIDTH] (E.7 SXTSY)
NO SCALE



AXIAL LOAD BEARING JAMB & HEAD
[>4'-0" (1.2m) OPENING WIDTH] (E.7 SXTSY)
NO SCALE

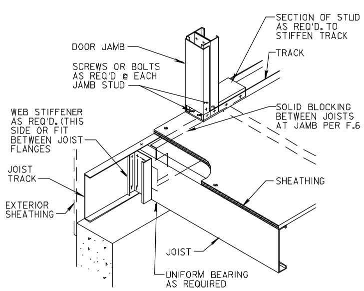


JAMB & SILL
[<4'-0" (1.2m) OPENING WIDTH] (E.8 SXTSY)
NO SCALE

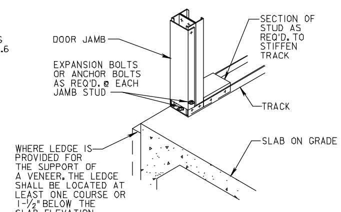


JAMB & SILL
[>4'-0" (1.2m) OPENING WIDTH] (E.9 SXTSY)
NO SCALE

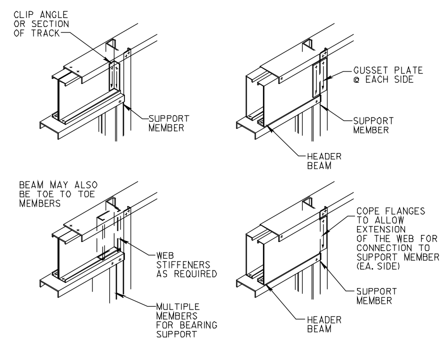
B



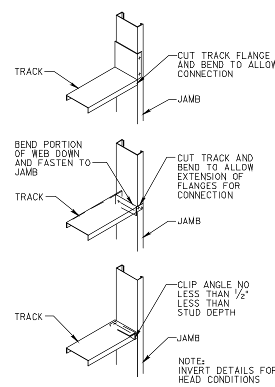
DOOR JAMB BASE AT FRAMING (E.10 SXTSY)
NO SCALE



DOOR JAMB BASE AT FRAMING (E.10 SXTSY)
NO SCALE



ALTERNATE HEADER BEAM CONNECTIONS (E.12 SXTSY)
NO SCALE



ALTERNATE SILL AND HEAD TRACK CONNECTIONS (E.13 SXTSY)
NO SCALE

A

FOR REVIEW ONLY

$$ – THINK VALUE ENGINEERING – $$

Revisions

Symbol	Descriptions	Date	Approved

US ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
OMAHA, NE

Designed by: X
Drawn by: X
Checked by: X
Reviewed by:
Submitted by:

AIR FORCE STANDARDS
PRESSURIZED HYDRANT FUELING SYSTEM
TYPE III

FRAMING DETAILS FOR FLOOR & WALL OPENINGS

Plot Scale Ratio: 4:1
Design File: STD01OCE\STLS106.DGN
Date:
Spec. No.:
Drawing Code:
Sheet reference number: S-6

COMPUTER AIDED DESIGN & DRAFTING

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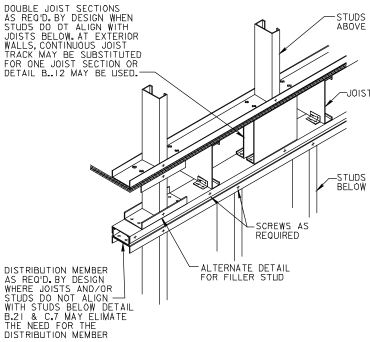
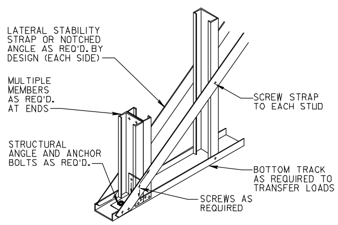
5 4 3 2 1

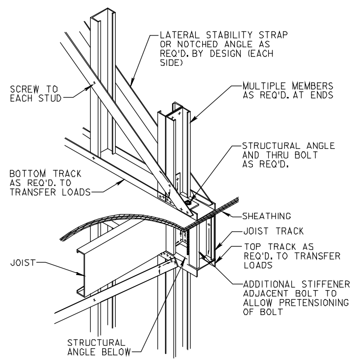


TOP TRACK DISTRIBUTION MEMBER F.1 SXISY
NO SCALE



NOTES:
STRAP FORCES MAY REQUIRE ADDITIONAL STIFFENING OF THE BOTTOM TRACK OR STRUCTURAL ANGLE.

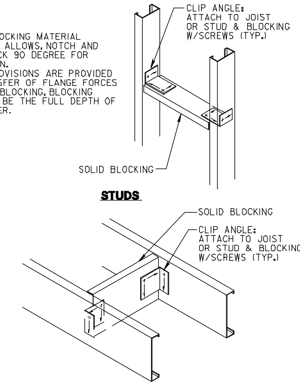
DIAGONAL STABILITY BRACING ANCHORAGE AT BASE F.2 SXISY
NO SCALE



DIAGONAL STABILITY BRACING AT FLOOR TRANSITION F.3 SXISY
NO SCALE

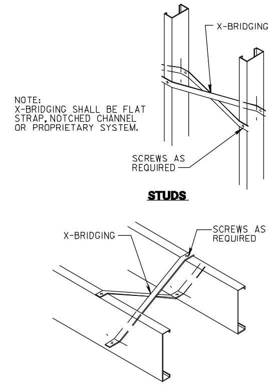
NOTE:
STRAP FORCES MAY REQUIRE ADDITIONAL STIFFENING OF TOP AND BOTTOM TRACK OR STRUCTURAL ANGLE.

NOTE:
1. WHERE BLOCKING MATERIAL THICKNESS ALLOWS, NOTCH AND BEND TRACK 90 DEGREE FOR CONNECTION.
2. WHERE PROVISIONS ARE PROVIDED FOR TRANSFER OF FLANGE FORCES TO SOLID BLOCKING, BLOCKING NEED NOT BE THE FULL DEPTH OF THE MEMBER.

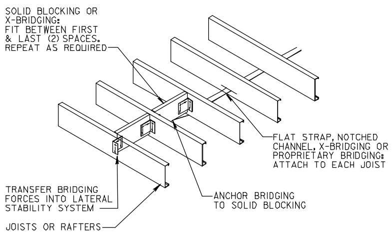


SOLID BLOCKING F.4 SXISY
NO SCALE

NOTE:
X-BRIDGING SHALL BE FLAT STRAP, NOTCHED CHANNEL OR PROPRIETARY SYSTEM.

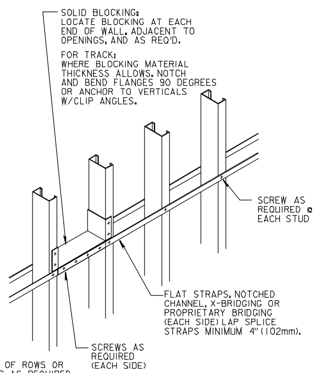


X-BRIDGING F.5 SXISY
NO SCALE



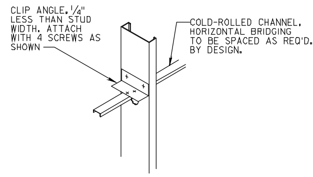
NOTE:
IF SHEATHING IS NOT INSTALLED ON MEMBERS, BRIDGING IS REQ'D. ON BOTH FLANGES.

JOIST AND RAFTER BRIDGING F.6 SXISY
NO SCALE

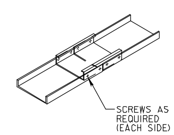


NOTE:
NUMBER OF ROWS OR BRIDGING AS REQUIRED BY DESIGN.

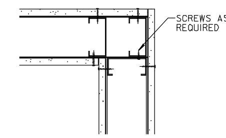
WALL BRIDGING F.7 SXISY
NO SCALE



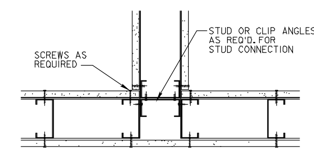
WALL BRIDGING (ALTERNATE) F.8 SXISY
NO SCALE



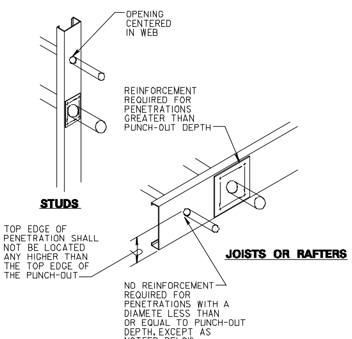
TOP & BOTTOM TRACK SPLICE F.9 SXISY
NO SCALE



CORNER FRAMING F.10 SXISY
NO SCALE

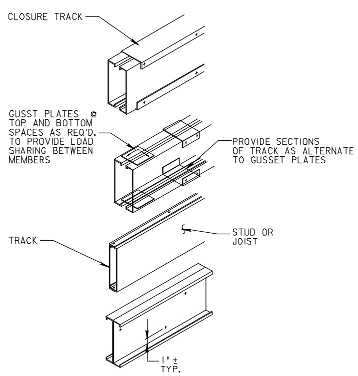


WALL INTERSECTION FRAMING F.11 SXISY
NO SCALE



NOTES:
1. FLANGES SHALL NOT BE NOTCHED OR CUT.
2. CAPACITY VERIFICATION BY DESIGN IS REQ'D. FOR ANY OPENINS LOCATED AT CONCENTRATED LOADS AND BEARING ENDS.
3. FOR UNPUNCHED MEMBERS CONSULT THE MANUFACTURER.

JOIST, STUD, OR RAFTER WEB PENETRATIONS F.12 SXISY
NO SCALE



MULTIPLE MEMBER CONFIGURATIONS F.13 SXISY
NO SCALE

FOR REVIEW ONLY

$$ – THINK VALUE ENGINEERING – $$

Revisions

Symbol	Descriptions	Date	Approved

US ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
OMAHA, NE

Designed by: X
Drawn by: X
Checked by: X
Reviewed by:
Submitted by:

AIR FORCE STANDARDS
PRESSURIZED HYDRANT FUELING SYSTEM
TYPE III

SPECIAL REINFORCEMENT BRIDGING, BLOCKING & MISC.

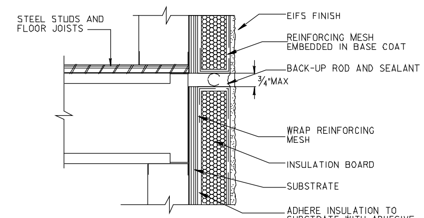
Plot Scale Ratio: 4:1
Design File: STD01OCE/STLS107.DGN
Spec. No.:
Date:
Drawing Code:
Sheet reference number: S-7

COMPUTER AIDED DESIGN & DRAFTING

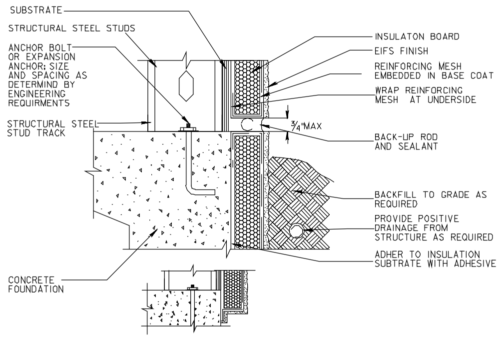
5 4 3 2 1

5 4 3 2 1

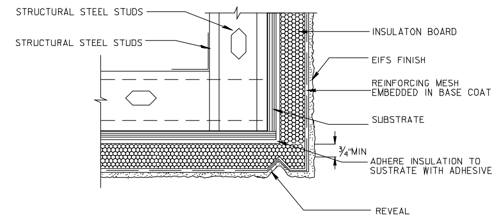
D



INTERMEDIATE LEVEL/WOOD FRAMING
EXPANSION JOINT AT FLOOR LEVEL C.9 SXISY
NO SCALE

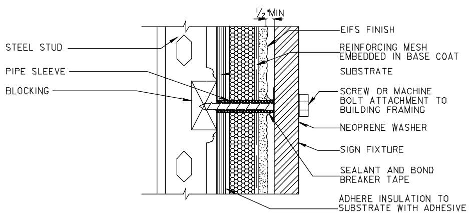


GRADE LEVEL/LIGHT GAUGE EXPANSION
JOINT FRAMING AT CONCRETE SLAB C.9 SXISY
NO SCALE

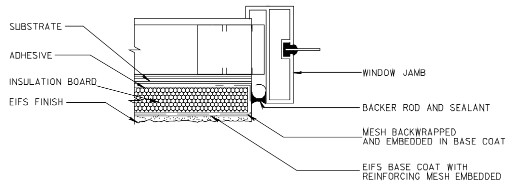


LIGHT GAUGE FRAMING
ROUTER CUT DRIP REGLET C.9 SXISY
NO SCALE

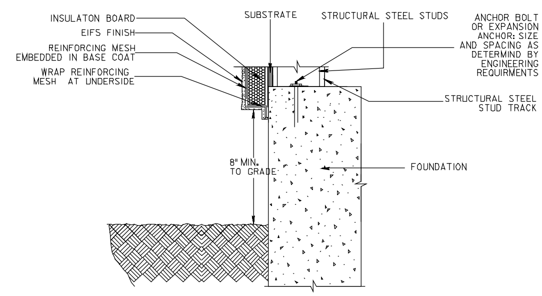
C



SIGN ATTACHMENT C.9 SXISY
NO SCALE

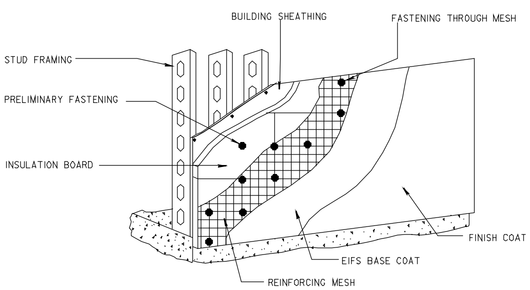


BACKWRAPPED JAMB C.9 SXISY
NO SCALE



T2.01 ABOVE GRADE C.9 SXISY
NO SCALE

B



G1.01 FRAMED WALL APPLICATION C.9 SXISY
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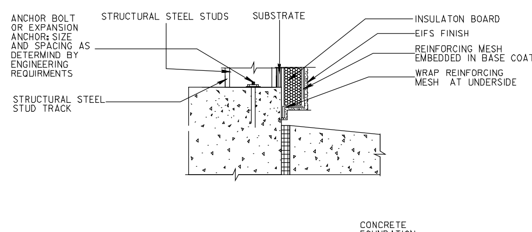

T1.01 ABOVE PAVEMENT C.9 SXISY
NO SCALE

A

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US ARMY ENGINEER DISTRICT
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Reviewed by:
Submitted by:
Chief

AIR FORCE STANDARDS
PRESSURIZED HYDRANT FUELING SYSTEM
TYPE III

SPECIAL REINFORCEMENT
BRIDGING, BLOCKING & MISC.

Plot Scale Ratio: 4:1
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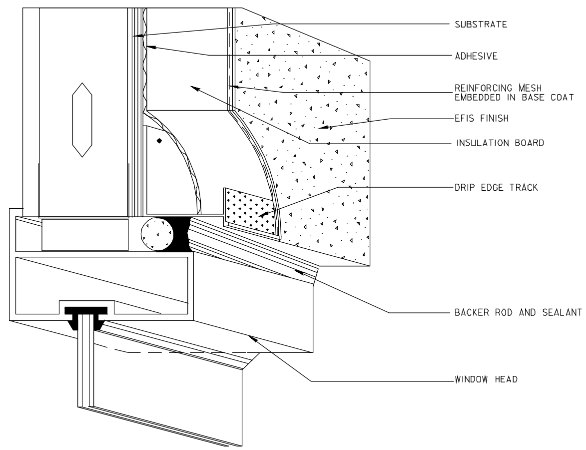
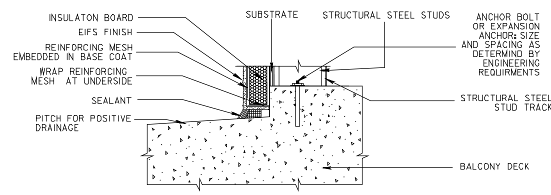


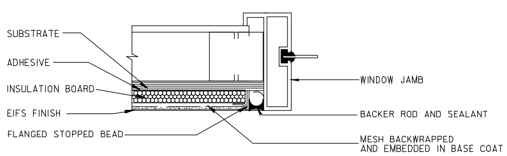
W5.01 HEAD WITH DRIP EDGE TRACK C.9 SXISY

NO SCALE



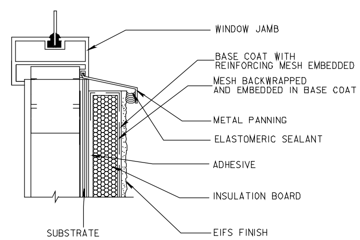
T6.01 BALCONY DECK C.9 SXISY

NO SCALE



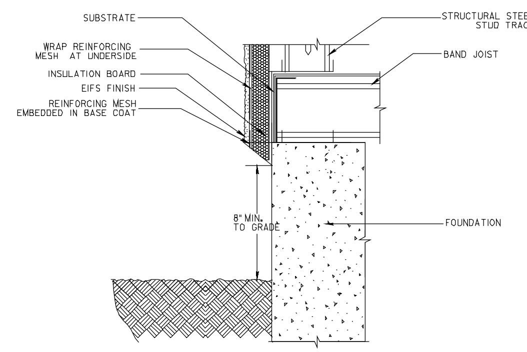
W7.01 JAMB TERMINATION C.9 SXISY

NO SCALE



W5.01 SILL TERMINATION AT PANNING C.9 SXISY

NO SCALE



T2.01 ABOVE GRADE C.9 SXISY

NO SCALE

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Revisions

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US ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
OMAHA, NE

Designed by: X

Drawn by: X

Checked by: X

Reviewed by:

Submitted by:

Chief: [redacted] Section

AIR FORCE STANDARDS
PRESSURIZED HYDRANT FUELING SYSTEM
TYPE III

SPECIAL REINFORCEMENT BRIDGING, BLOCKING & MISC.

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