STEEL DECK INSTITUTE

AMERICAN NATIONAL STANDARDS INSTITUTE/ STEEL DECK INSTITUTE

C - 2017 Standard for

Composite Steel Floor Deck-Slabs

Approved American National Standard

ANSI

1. General

1.1 Scope:

A. This Standard for Composite Steel Floor Deck-Slabs, hereafter referred to as the Standard, shall govern the materials, design, and erection of composite concrete slabs utilizing cold formed steel deck functioning as a permanent form and as reinforcement for positive moment in floor and roof applications in buildings and similar structures.

B. The Appendices shall be part of the Standard.

C. The User Notes and Commentary shall not be part of the Standard.

User Note: User Notes and Commentary are intended to provide practical guidance in the use and application of this Standard.

D. Where the Standard refers to “designer,” this shall mean the entity that is responsible to the Owner for the overall structural design of the project, including the steel deck.

User Note: This is usually the Structural Engineer of Record.

E. Equations that appear in this Standard are compatible with the US Customary System (USCS) of units. However, any consistent system of units shall be permitted to be used. SI units or equations shown in parenthesis in this Standard are for information only and are not part of this Standard.

User Note: The USCS is also referred to as English Units or the inch-pound system.

F. Terms not defined in this Standard, AISI S100 or AISI/AISC shall have the ordinary accepted meaning for the context for which they are intended.

G. It shall be permitted to specify deck base steel thickness either by dimensional thickness, or by gage when the relationship of base steel thickness to gage has been defined by the deck manufacturer. However, for the purpose of design, the dimensional thickness shall be used.

User Note: Both AISI and SDI now specify steel thickness in terms of design thickness in lieu of gage thickness. Gage thicknesses, however, are still commonly referred to in the metal deck industry. Table UN-1.1 shows common gages and corresponding uncoated design and minimum steel thicknesses.

Table UN-1.1

Gage No.	Design Thickness		Minimum Thickness[1]	
	in.	mm.	in.	mm.
22	0.0295	0.75	0.028	0.71
20	0.0358	0.91	0.034	0.86
18	0.0474	1.20	0.045	1.14
16	0.0598	1.52	0.057	1.44

[1] Minimum delivered thickness is 95% of the design thickness.

H. Except as specifically required by this Standard, ACI 318 shall not be applicable to the design or construction of composite steel deck-slabs.

User Note: Refer to Section 1.4.9 of ACI 318.

1.2 Reference Codes, Standards, and Documents:

A. Codes and Standards: The following documents or portions thereof are referenced in this Standard and shall be considered part of the requirements of this Standard. Where these documents conflict with this Standard, the requirements of this Standard shall control:

:

1. American Concrete Institute (ACI)
   a. ACI 318-14, Building Code Requirements for Structural Concrete
2. American Iron and Steel Institute (AISI)
   a. AISI S100-16, North American Specification for the Design of Cold-Formed Steel Structural Members.
   b. AISI S310-16, North American Standard for the Design of Profiled Steel Diaphragm Panels.
   c. AISI S905-13, Test Methods for Mechanically Fastened Cold-Formed Steel Connections
   d. AISI/AISC, Standard Definitions for Use in the Design of Steel Structures, 2007 edition
3. American Institute of Steel Construction (AISC)
   a. ANSI/AISC 360-16, Specification for Structural Steel Buildings
4. American Society for Testing and Materials (ASTM)
   a. ASTM A820 / A820M – 11, Standard Specification for Steel Fibers for Fiber-Reinforced Concrete
   b. ASTM A1008 / A1008M - 15, Standard Specification for Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy with Improved Formability, Solution Hardened, and Bake Hardenable
   c. ASTM C1116 / C1116M - 10a(2015) Standard Specification for Fiber-Reinforced Concrete
   d. ASTM D7508 / D7508M - 10 Standard Specification for Polyolefin Chopped Strands for Use in Concrete
5. American Society of Civil Engineers (ASCE)
   a. SEI/ASCE 7-16, Minimum Design Loads for Buildings and Other Structures
6. American Welding Society (AWS)
   a. AWS D1.1:2015, Structural Welding Code-Steel
   b. AWS D1.3:2008, Structural Welding Code-Sheet Steel
7. Steel Deck Institute (SDI)
   a. SDI-T-CD-2017, Test Standard for Composite Steel Deck-Slabs

Commentary: The following documents are referenced within the Commentary and User Notes:

1. American Association of State Highway and Transportation Officials (AASHTO)
   a. AASHTO LRFD Bridge Design Specifications, Customary U.S. Units, 7th Edition, with 2015 Interim Revisions
2. American Concrete Institute (ACI)
   a. ACI 215R-92, Considerations for Design of Concrete Structures Subjected to Fatigue Loading
   b. ACI 302.1R-04, Guide for Concrete Floor and Slab Construction
   c. ACI 224.1R-07, Causes, Evaluation, and Repair of Cracks in Concrete Structures
   d. ACI 318-14, Building Code Requirements for Structural Concrete
   e. ACI 544.3R-08, Guide for the Specification, Proportioning and Production of Fiber Reinforced Concrete
   f. ACI Concrete Terminology, http://terminology.concrete.org
3. American Institute of Steel Construction (AISC)
   a. AISC Design Guide No. 11, Floor Vibrations Due to Human Activity, 1997
   b. AISC Design Guide No. 18, Steel-Framed Open-Deck Parking Structures, 2003
   c. ANSI/AISC 360-16, Specification for Structural Steel Buildings
4. American Iron and Steel Institute (AISI)
   a. AISI S100-16, North American Specification for the Design of Cold-Formed Steel Structural Members.
5. American Society for Testing and Materials (ASTM)
   a. ASTM A653 / A653M - 15 Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process
   b. ASTM A1008 / A1008M - 15, Standard Specification for Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy with Improved Formability, Solution Hardened, and Bake Hardenable
   c. ASTM E119 - 14, Standard Test Methods for Fire Tests of Building Construction and Materials
6. Steel Deck Institute (SDI)
   a. SDI COSP-2017, Code of Standard Practice.
   b. SDI-DDM, Diaphragm Design Manual, 4th Edition.
   c. SDI FDDM, Floor Deck Design Manual, 1st Edition.
   d. SDI-MOC, Manual of Construction with Steel Deck, 3rd Edition
   e. SDI Position Statement “Use of Composite Steel Floor Deck in Parking Garages.”
7. Underwriters Laboratories (UL)
   a. Fire Resistance Directory

1.3 Construction Documents

Construction Documents: The construction documents shall describe the composite slabs that are to be constructed and shall include not less than the following information:

A. Loads
   1. Composite slab loads as required by the applicable building code. Where applicable, load information shall include concentrated loads.
   2. Assumed construction phase loads.

B. Structural framing plans for all composite slabs showing the size, location and type of all deck supports.

C. Deck and Deck Attachment
   1. Depth, type (profile), and design thickness.
   2. Deck material (including yield strength) and deck finish,
   3. Deck attachment type, spacing, and details.

D. Concrete and Reinforcing
   1. Specified concrete strength, f'_c
   2. Specified concrete density (and tolerance if required for fire rating assembly)
   3. Specified strength or grade of reinforcing steel or welded wire reinforcement (if used)
   4. Size, extent and location of all reinforcement (if used)
   5. Slab thicknesses (and tolerance if required for fire rating assembly)
   6. Discontinuous fiber reinforcement material, type and dosage (if used).

> **User Note:** The following is an example of a composite slab as it could be specified on the contract drawings: "Composite slab shall consist of 1-1/2 inch deep G-60 galvanized composite steel deck, design thickness 0.0358 inch (20 gage), F_y = 50 ksi (Type XX by YY, Inc or approved equivalent) with 3 inch thick, 3000 psi, normal-weight concrete topping (total thickness = 4-1/2 inches) reinforced with XXX. Deck shall be attached to supporting framing using #12 screws in a 36/7 pattern. (4) #10 sidelap screws shall be installed per deck span."

2. Products

2.1 Material:

A. Sheet steel for deck shall conform to AISI S100, Section A3, as modified in Section 2.1.C.

> **User Note:** AISI S100, Section A3.1 notes an exception that "For steels used in composite slabs, the requirements of SDI-C shall be followed exclusively." The intention of this exception is to permit that the requirements of Sections 2.1.C.2 and 2.1.C.3 shall be followed regarding limits on design yield stress and ductility.

B. Sheet steel for accessories that carry defined loads shall conform to AISI S100, Section A3. Sheet steel for non-structural accessories that do not carry defined loads shall be permitted to be any steel that is adequate for the proposed application.

C. All sheet steel used for deck or accessories that carry defined loads shall have a minimum specified yield stress that meets or exceeds 33 ksi (230 Mpa).

1 For the case where the steel deck acts as a form, design yield and tensile stresses shall be determined in accordance with AISI S100, Section A3.

2. For the case where the steel deck acts as tensile reinforcement for the composite deck-slab, the steel shall conform to AISI S100, Section A3. When the ductility of the steel measured over a two-inch (50 mm) gage length is 10% or greater, the maximum design yield stress shall not exceed the lesser of 50 ksi (345 Mpa) or F_y. When the ductility of the steel measured over a two-inch (50 mm) gage length is less than 10%, the maximum design yield stress shall not exceed the lesser of 50 ksi (345 Mpa) or 0.75 F_y.

3. When the ductility of the steel used for deck, measured over a two-inch (50 mm) gage length, is less than 10%, the ability of the steel to be formed without cracking or splitting shall be demonstrated.

Commentary: Most steel deck is manufactured from steel conforming to ASTM A1008 /A1008M, Structural Sheet for uncoated or uncoated top/painted bottom deck or from ASTM A653 / A653M, Structural Sheet for galvanized deck. In most cases the designer will choose one finish or the other. However, both types of finish may be used on a project, in which case the designer must indicate on the plans and project specifications the areas in which each is used. (Refer to Section 2.3 of this Standard). Stainless steel is not recommended due to the lack of available performance data.

D. Concrete and Reinforcement:

1. Concrete placed on steel deck shall conform to ACI 318, Chapter 19, except as modified by Sections 2.1.D.2 and 2.1.D.3.

2. The specified concrete compressive strength shall not be less than 3000 psi (21 MPa). The maximum compressive strength used to calculate the strength of the composite deck-slab shall not exceed 6000 psi (42 MPa).

User Note: Load tables and labeled fire resistant rated assemblies may require concrete compressive strengths in excess of 3000 psi. The average compressive strength of the concrete may exceed 6000 psi, but a maximum strength of 6000 psi is to be used in calculating the strength of the composite deck-slab.

3. Admixtures containing chloride salts or other substances that are corrosive or otherwise deleterious to the steel deck and embedded items shall not be permitted.

4. Steel Reinforcing shall conform to ACI 318, Section 20.2

5. Discontinuous fiber reinforcement shall conform to the following:
   a. Steel fibers: ASTM A820.
   b. Macrosynthetic fibers: ASTM D7508.

2.2 Tolerance of Delivered Material:

A. The minimum uncoated steel thickness as delivered to the job site shall not at any location be less than 95% of the design thickness, however lesser thicknesses shall be permitted at bends, such as corners, due to cold-forming effects.

User Note: The minimum delivered thickness is in accordance with AISI S100.

B. Panel length shall be no less than ½ inch (13 mm) shorter than the specified length nor greater than ½ inch (13 mm) longer than the specified length for single span or butted end deck. Panel length shall be no less than ½ inch (13 mm) shorter than the specified length for lapped end deck.

User Note: No restriction is placed on over length panels in lapped applications because there is no adverse consequence in this application.

C. Panel cover width shall be no less than 3/8 inch (10 mm) less than the specified panel width, nor more than 3/4 inch (19 mm) greater than the specified width.

D. Panel camber and/or sweep shall not be greater than 1/4 inch in a 10 foot length (6 mm in 3 m).

E. Panel end out of square shall not exceed 1/8 inch per foot of panel width (10 mm per m).

F. Embossments (if used) shall not be less than 90% of the design embossment depth.

2.3 Finish:

A. The finish on the steel deck shall be specified by the designer.

B. Galvanizing or other metallic coatings (if specified by the designer) shall conform to the requirements of the applicable steels in AISI S100, Section A3.

User Note: The most commonly specified galvanized sheet steel is ASTM A653 / A653M.

C. A shop coat of primer paint (bottom side only) shall be applied to steel sheet if specified by the designer.

Commentary: The finish on the steel composite deck must be specified by the designer and be suitable for the environment to which the deck is exposed within the finished structure. Because the composite deck is the positive bending reinforcement for the slab, its service life should at least be equal to the design service life of the structure. Zinc-Aluminum finishes are not recommended. When composite deck with an unpainted top and painted bottom is used, the primer coat is intended to protect the steel for only a short period of exposure in ordinary atmospheric conditions and shall be considered an impermanent and provisional coating. In highly corrosive or chemical atmospheres or where reactive materials could be in contact with the steel deck, special care in specifying the finish should be used, which could include specialized coatings or materials. If specifying painted deck in areas that require spray-on fireproofing, the paint must be permitted by the applicable fire rated assembly. Not all paints are approved for fire rated assemblies. This requirement must be clearly called out in the contract documents. In general, there are three types of fire resistive assemblies; those achieving the fire resistance by membrane protection, direct applied protection, or with an unprotected assembly. Of these three, only the systems that utilize direct applied protection are concerned with the finish of the steel deck. In these systems, the finish of the steel deck can be the factor that governs the fire resistance rating that is achieved. In assemblies with direct applied fire protection the finish (paint) is critical.

In the Underwriters Laboratories Fire Resistance Directory, some deck manufacturing companies have steel deck units that are classified in some of the D700, D800, and D900-series concrete and steel floor units. These classified deck units (Classified Steel Floor and Form Units) are shown as having a galvanized finish or a phosphatized/painted finish. These classified deck units have been evaluated for use in these specific designs and found acceptable.

2.4 Design:

A. Deck as a form

1. Design by either Allowable Strength Design (ASD) or Load and Resistance Factor Design (LRFD) shall be permitted. The section properties and allowable strength (ASD) or design strength (LRFD) for the steel deck shall be computed in accordance with AISI S100.
2. Deck shall be evaluated for strength under the following load combinations:
   a. Allowable Strength Design

$$w_{dc} + w_{dd} + w_{lc} \qquad \text{(Eq. 2.4.1)}$$

$$w_{dc} + w_{dd} + P_{lc} \qquad \text{(Eq. 2.4.2)}$$

$$w_{dd} + w_{cdl} \qquad \text{(Eq. 2.4.3)}$$

Where:

w_{dc} = dead weight of concrete

w_{dd} = dead weight of the steel deck

w_{lc} = uniform construction live load (combined with fluid concrete) not less than 20 psf (0.96 kPa)

w_{cdl} = uniform construction live load (combined with bare deck), not less than 50 psf (2.40 kPa)

P_{lc} = concentrated construction live load per unit width of deck section, 150 pounds on a 1 foot width (2.19 kN on a 1 meter width)

User Note: The uniform construction live load of 20 psf is considered adequate for typical construction applications that consist of concrete transport and placement by hose and concrete finishing using hand tools. The designer typically has little control over means-and-methods of construction, and should bring to the attention of the constructor that bulk dumping of concrete using buckets, chutes, or handcarts, or the use of heavier motorized finishing equipment such as power screeds, may require design of the deck as a form using uniform construction live loads, w_{lc} ,of 50 psf or greater. Section A1.3 requires that the designer include the assumed construction loads in the construction documents and it is suggested that the constructions documents require verification of adequacy by the constructor.

User Note: The designer should account for additional loads attributable to concrete ponding due to deflections of the structural system, including deck and support framing. See SDI-FDDM for additional information.

b. Load and Resistance Factor Design

$1.6w_{dc} + 1.2w_{dd} + 1.4w_{lc}$ (Eq. 2.4.4)

$1.6w_{dc} + 1.2w_{dd} + 1.4P_{lc}$ (Eq. 2.4.5)

$1.2w_{dd} + 1.4w_{cdl}$ (Eq. 2.4.6)

Commentary: The load factor used for the dead weight of the concrete is 1.6 because of delivering methods and an individual sheet can be subjected to this load. The use of a load factor of 1.4 for construction load in LRFD design is calibrated to provide equivalent design results in ASD design. Refer to the commentary of AISI S100 for additional information.

3. Cantilever spans shall be evaluated for strength under the following load combinations:
   a. Allowable Strength Design: Equations 2.4.1 and 2.4.2 shall be applied to both the cantilever span and the adjacent span. The concentrated construction live load (P_{lc}) shall be applied at the end of the cantilever.
   b. Load and Resistance Factor Design: Equations 2.4.4 and 2.4.5 shall be applied to both the cantilever span and the adjacent span. The concentrated construction live load (P_{lc}) shall be applied at the end of the cantilever.
4. Special loading considerations:
   a. The specified construction live loads shall be increased when required by the construction operations.
   b. Loads shall be applied in a sequence that simulates the placement of the concrete, in accordance with Appendix 1. Rational analysis shall be permitted to be used for developing shear and moment diagrams and calculating deflections for non-uniform spans.

Commentary: The loading shown in Figure 1 of Appendix 1 is representative of the sequential loading of fresh concrete on the deck. The 150 pound per foot of width (2.19 kN per 1 m of width) load is the equivalent of distributing a 300 pound (1.33 kN) worker over a 2 foot (600 mm) width. Experience has shown this to be a conservative distribution.

Single span deck conditions have no redundancy because they are statically determinate, as opposed to multi-span conditions that are statically indeterminate. Because of this lack of redundancy, additional consideration should be given to proper specification of construction live and dead loads. Allowable construction spans for single-span deck may be shorter than for multi-span applications, and the designer must consider this in locations where it is impossible to install the deck in a multi-span condition, such as between stair and elevator towers. Whenever possible, the deck should be designed as a multi-span system that does not require shoring during concrete placement.

The specified construction live loads reflect nominal loads from workers and tools and do not include loads of equipment such as laser screeds or power trowels nor additional concrete weight due to ponding. If anticipated construction activities include these additional loads, they should be considered in the design.

5. Deck Deflection
   a. Calculated deflections of the deck as a form shall be based on the load of the concrete as determined by the design slab thickness and the self-weight of the steel deck, uniformly loaded on all spans. Deflections shall be limited to the lesser of 1/180 of the clear span or 3/4 inch (19 mm). Calculated deflections shall be relative to supporting members.
   b. The deflection of cantilevered deck as a form, as determined by slab thickness and self-weight of the steel deck, shall not exceed a/90, where "a" is the cantilever length, nor 3/4 inches (19 mm).

Commentary: The deflection calculations do not take into account construction loads because these are considered to be temporary loads. The deck is designed to always be in the elastic range, so removal of temporary loads will allow the deck to recover, unless construction overloads cause the stress in the deck to exceed the elastic limits of the deck. The supporting structural steel also deflects under the loading of the concrete.

The designer is urged to check the deflection of the total system. Typical load tables are based on uniform slab thickness. If the designer wants to include additional concrete loading on the deck because of frame deflection, the additional load should be shown on the design drawings or stated in the deck section of the contract documents.

6. Minimum Bearing and Edge Distance: Minimum bearing lengths and fastener edge distances shall be determined in accordance with AISI S100.

User Note: Figure 2 in Appendix 1 indicates support reactions. The designer should check the deck web crippling capacity based on available bearing length. The deck should be adequately attached to the structure to prevent the deck from slipping off the supporting structure.

7. Diaphragm Shear Capacity: Diaphragm strength and stiffness shall be determined utilizing the bare steel deck capacity without concrete in accordance with AISI S310.

User Note: SDI-DDM contains diaphragm load tables that comply with AISI S310.

8. Connections: Deck shall be attached to supports to resist loads and to provide structural stability for the supporting member. Connections shall be designed in accordance with AISI S100 or strengths shall be determined by testing in accordance with AISI S905. Tests shall be representative of the design. When tests are used and the design allows either end laps or single thickness conditions, both conditions shall be tested.

B. Deck and Concrete as a Composite Slab:

1. Design by either Allowable Strength Design (ASD) or Load and Resistance Factor Design (LRFD) shall be permitted.

Commentary: The ASD Factor of Safety is calibrated to the LRFD resistance factor at a live to dead load ratio of 3 to 1, which is a reasonable load ratio for composite deck slabs and matches the ratio used in the AISC 360 Standard.

2. Flexural Resistance: Flexural resistance of the composite deck-slab shall be determined in accordance with one of the following methods:
   a. “Prequalified Section Method” as per Appendix 2.
   b. “Shear Bond Method” as per Appendix 3.
   c. "Ultimate Strength Method" as per Appendix 4.
   d. Full scale performance testing as per SDI-T-CD.
   e. Other methods approved by the authority having jurisdiction.
3. Resistance to limit states other than flexure shall be determined in accordance with the other provisions of Section 2.4B.
4. Deck shall be evaluated for strength under the load combinations required by the applicable building code. In the absence of a building code, the load combinations prescribed by ASCE 7 shall be used.
5. Load Determination: The superimposed load capacity shall be determined by deducting the weight of the slab and the deck from the total load capacity. Unless composite deck-slabs are designed for continuity, slabs shall be assumed to act on simple spans.

Commentary: Most published live load tables are based on simple span analysis of the composite system; that is, a continuous slab is assumed to crack over each support and to carry load as a series of simple spans.

Commentary: By using the reference analysis techniques or test results, the deck manufacturer determines the live loads that can be applied to the composite deck-slab combination. The results are usually published as uniform load tables. For most applications, the deck thickness and profile is selected so that shoring is not required; the live load capacity of the composite system is usually more than adequate for the superimposed live loads. In calculating the section properties of the deck, AISI S100 may require that compression zones in the deck be reduced to an “effective width,” but as tensile reinforcement, the total area of the cross section may be used. (See Appendix 5)

Coatings other than those tested may be investigated, and if there is evidence that their performance is better than that of the tested product, additional testing may not be required.

6. Concrete: Specified concrete compressive strength (f’c) shall comply with Section 2.1 and shall not be less then 3000 psi (21 MPa), nor less than that required for fire resistance ratings or durability.

Commentary: Load tables are generally calculated by using a concrete strength of 3000 psi (21 MPa). Composite slab capacities are not greatly affected by variations in concrete compressive strength; but if the strength falls below 3000 psi (21 MPa), it would be advisable to check shear anchor design for composite beam action.

a. Minimum Cover: The concrete thickness above the top of the steel deck shall not be less than 2 inches (50 mm), nor that required by any applicable fire resistance rating requirements. Minimum concrete cover for reinforcement shall be in accordance with ACI 318 Section 20.6.1.

7. Deflection: Deflection of the composite slab shall be in accordance with the requirements of the applicable building code.
   a. Cross section properties shall be calculated in accordance with Appendix 5.
   b. Additional deflections resulting from concrete creep, where applicable, shall be calculated by multiplying the immediate elastic deflection due to the sustained load by the following factors:
      i. (1.0) for load duration of 3 months
      ii. (1.2) for load duration of 6 months
      iii. (1.4) for load duration of 1 year
      iv. (2.0) for load duration of 5 years.

Commentary: Live load deflections are seldom a controlling design factor. A superimposed live load deflection of span/360 is typically considered to be acceptable. The deflection of the slab/deck combination can be predicted by using the average of the cracked and uncracked moments of inertia as determined by the transformed section method of analysis. Refer to Appendix 5 of this Standard or SDI-FDDM.

Commentary: Limited information on creep deflections is available. This method is similar to the procedure for reinforced concrete slabs. Because the steel deck initially carries the weight of the concrete when constructed without shoring, only the superimposed loads should be considered when creep deflections are a concern. When shoring is used, the weight of the concrete should be considered in the loads which contribute to creep.

Commentary: Floor vibration performance is the result of the behavior of entire floor system, including the support framing. The designer should check vibration performance using commonly accepted methods, which may include AISC Design Guide No. 11.

8. Special Loads: The following loads shall be considered in the analysis and calculations for strength and deflection:
   a. Suspended Loads.
   b. Concentrated Loads
   c. Moving Loads
   d. Cyclic Loads

9. One-way Shear Strength: This section shall be used to determine the one-way shear strength of the composite deck-slab.

LRFD:

$$\phi V_n = \phi_v V_c + \phi_s V_D \le \frac{\phi_v 4\sqrt{f_c'}A_c}{1000} \qquad \text{(Eq. 2.4.7a) (USCS)}$$

$$\phi V_n = \phi_v V_c + \phi_s V_D \le \phi_v 0.332\sqrt{f_c'}A_c \qquad \text{(Eq. 2.4.7b) (SI)}$$

ASD:

$$V_n/\Omega = V_c/\Omega_v + V_D/\Omega_s \le \frac{4\sqrt{f_c'}A_c/\Omega_v}{1000} \qquad \text{(Eq. 2.4.7c) (USCS)}$$

$$V_n/\Omega = V_c/\Omega_v + V_D/\Omega_s \le 0.332\sqrt{f_c'}A_c/\Omega_v \qquad \text{(Eq. 2.4.7d) (SI)}$$

Where:

$$V_c = 2\lambda\sqrt{f_c'}A_c \qquad \text{(Eq. 2.4.8a) (USCS)}$$

$$V_c = 0.166\lambda\sqrt{f_c'}A_c \qquad \text{(Eq. 2.4.8b) (SI)}$$

V_D = nominal shear strength of the steel deck section calculated in accordance with AISI S100, kips (kN)
A_c = concrete area available to resist shear, in^2 (mm^2), see Figure 2-1.
f'_c = specified compressive strength of concrete, psi (MPa)
λ = 1.0 where concrete density exceeds 130 lbs/ft^3 (2100 kg/m^3); 0.75 where concrete density is equal to or less than 130 lbs/ft^3 (2100 kg/m^3).
ϕ_v = 0.75
ϕ_s = 0.85
Ω_v = 2.00
Ω_s = 1.75

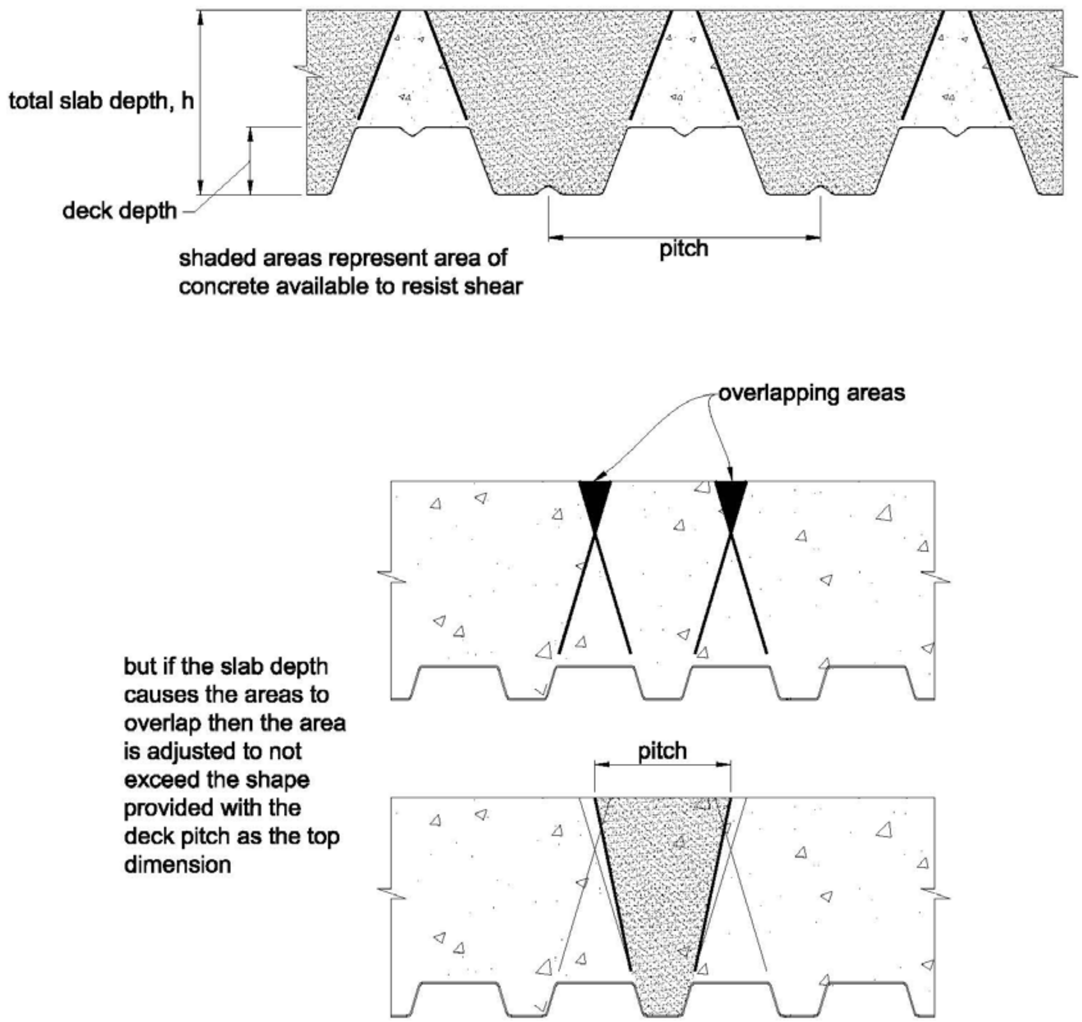


Figure 2-1 One-Way Shear Parameters

User Note: The area of the concrete, Ac defined in Figure 2-1 is relative to the pitch width of the steel deck. The vertical shear strength of two webs of the steel deck should be used in combination with Ac in Equation 2.4.7a or b. The vertical shear strength is then for the pitch with of the deck. This may be converted to a unit width basis by multiplying by the ratio of the unit width to the pitch width of the steel deck.

10. Punching Shear Resistance: The critical surface for calculating punching shear shall be perpendicular to the plane of the slab and located outside of the periphery of the concentrated load or reaction area. The punching shear resistance, V_{pr}, shall be determined as follows:

LRFD

$$V_{pr} = \frac{(2+4/\beta_c)\phi v\sqrt{f'_c}\,b_o h_c}{1000} \le \frac{4\phi_v\sqrt{f_c'}\,b_o h_c}{1000} \qquad \text{(Eq. 2.4.9a)(USCS)}$$

$$V_{pr} = 0.043(2+4/\beta_c)\phi_v\sqrt{f'_c}\,b_o h_c \le 0.332\phi_v\sqrt{f_c'}\,b_o h_c \qquad \text{(Eq. 2.4.9b) (SI)}$$

ASD

$$V_{pr} = \frac{(2+4/\beta_c)\sqrt{f'_c}\,b_o h_c / \Omega_v}{1000} \le \frac{4\sqrt{f'_c}\,b_o h_c / \Omega_v}{1000} \qquad \text{(Eq. 2.4.9c) (USCS)}$$

$$V_{pr} = 0.043(2+4/\beta_c)\sqrt{f'_c}\,b_o h_c / \Omega_v \le 0.332\sqrt{f'_c}\,b_o h_c / \Omega_v \qquad \text{(Eq. 2.4.9d) (SI)}$$

Where:

b_o = perimeter of critical section, in. (mm).
h_c = thickness of concrete cover above steel deck, in. (mm)
V_{pr} = punching shear resistance, kips (kN)
β_c = ratio of long side to short side of concentrated load or reaction area
ϕ_v = 0.75
Ω_v = 2.00

The critical section shall be located so that the perimeter b_o is a minimum but need not be closer than $h_c/2$ to the periphery of the concentrated load or reaction area.

11. Concentrated Loads: Concentrated loads shall be permitted to be laterally distributed perpendicular to the deck ribs in accordance with this section. Alternate lateral load distributions based on rational analysis shall be permitted when allowed by the authority having jurisdiction.

a. Concentrated loads shall be distributed laterally (perpendicular to the ribs of the deck) over an effective width, b_e. The load distribution over the effective width, b_e, shall be uniform.

b. The concrete above the top of steel deck shall be designed as a reinforced one-way concrete slab in accordance with ACI 318 Chapter 7, transverse to the deck ribs, to resist the weak axis moment, M_{wa}, over a width of slab equal to W. Appropriate load and resistance factors as required by ACI 318 shall be applied to the weak axis moment.

$b_m = b_2 + 2\,t_c + 2\,t_t$ (Eq. 2.4.10)

$b_e = b_m + (2)(1\text{-}x/L)x \leq 106.8\,(t_c/h)$ for single span bending (Eq. 2.4.11)

$b_e = b_m + (4/3)(1\text{-}x/L)x \leq 106.8\,(t_c/h)$ for continuous span bending when reinforcing steel is provided in the concrete to develop negative bending. (Eq. 2.4.12)

$b_e = b_m + (1\text{-}x/L)x \leq 106.8\,(t_c/h)$ for shear (Eq. 2.4.13)

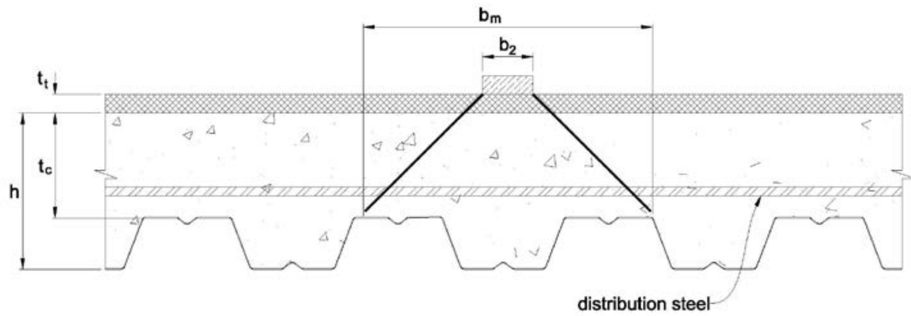
$W = L/2 + b_3 \leq L$ (Eq. 2.4.14)

$M_{wa} = 12\,P\,b_e\,/\,(15W)$ in-lb per foot (Eq. 2.4.15a) (USCS)

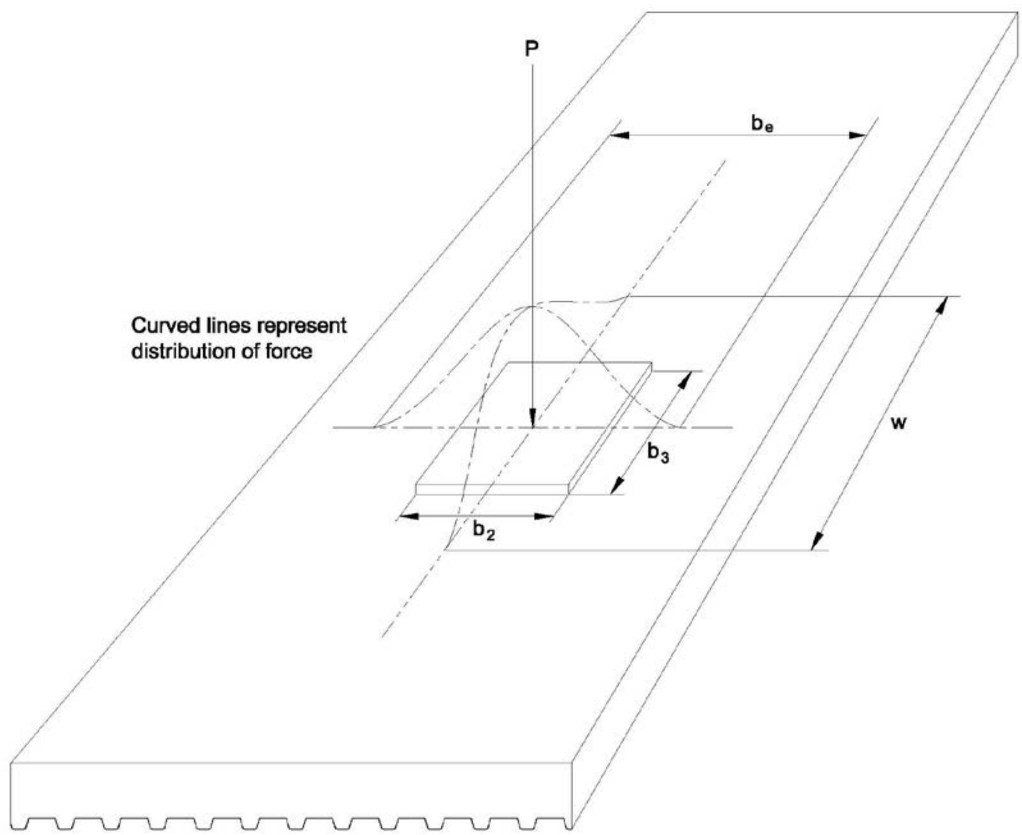
$= P\,b_e\,/\,(15\,W)$ N-mm per mm (Eq. 2.4.15b) (SI)

Where:

b_e = Effective width of concentrated load, perpendicular to the deck ribs, in (mm)
b_m = Projected width of concentrated load, perpendicular to the deck ribs, measured at top of steel deck, in (mm)
b_2 = Width of bearing perpendicular to the deck ribs, in (mm)
b_3 = Length of bearing parallel to the deck ribs, in (mm)
h = Depth of composite deck-slab, measured from bottom of steel deck to top of concrete slab, in (mm)
L = Deck span length, measured from centers of supports, in (mm)
M_{wa} = Weak axis bending moment, perpendicular to deck ribs, of width, in.-lbs per foot of width, (N-mm per mm of width)
P = Magnitude of concentrated load, lbs (N)
t_c = Thickness of concrete above top of steel deck, in (mm)
t_t = Thickness of rigid topping above structural concrete (if any), in (mm)
W = Effective length of concentrated load, parallel to the deck ribs, in (mm)
x = Distance from center of concentrated load to nearest support, in (mm)



User Note Figure 2-2



User Note Figure 2-3

User Note: User Note Figures 2-2 and 2-3 illustrate the dimensions associated with this section.

Commentary: The designer should take into account the sequence of loading. Suspended loads may include ceilings, light fixtures, ducts or other utilities. The designer should be informed of any loads to be applied after the composite slab has been installed. Care should be used during the placement of suspended loads on all types of hanger tabs or other hanging devices for the support of ceilings so that an approximate uniform loading is maintained. The individual manufacturer should be consulted for allowable loading on single hanger tabs. Improper use of hanger tabs or other hanging devices could result in the overstressing of tabs and/or the overloading of the composite deck-slab.

Commentary: Composite floor deck is not recommended as the only concrete reinforcement for use in applications where the floor is loaded with repeated lift truck (forklift) or similar heavy wheeled traffic. (Lift trucks are defined as small power operated vehicles that have devices for lifting and moving product. The definition of lift trucks does not include manually operated "pallet jacks.") Loading from lift trucks includes not only moving gravity loads, but also includes vertical impact loading and in-plane loading effects from starting, stopping, and turning. The repetitive nature of this loading, including impact, fatigue, and in-plane effects can be more detrimental to the slab-deck performance than the gravity loads. Suspended floor slabs subjected to lift truck traffic have special design requirements to ensure the fatigue stress in the reinforcement is low to keep the cracks sufficiently tight and serviceable to minimize crack spalling due to the hard wheel traffic. The design should only use the steel deck as a stay-in-place form. Structural concrete design recommendations contained in ACI 215R and AASHTO-LRFD are suggested for guidance in the design of these slabs. Due consideration for the stiffness of the supporting framing should be given by the designer.
Composite floor deck has successfully been used in applications that are loaded by occasional "scissor lift" use, and in warehouses with industrial racks without lift truck traffic and in areas serviced by "pallet jacks." Proper analysis and design for moving and point loads must be performed.
Closely spaced concentrated loads can cause overlapping influence zones which can result in increased loading within the effective width of a single load. The effects of there overlapping influence zones must be considered.

Commentary: Composite floor deck has been used in parking structures and other similar areas loaded by automobiles; however, the following precautions should be observed:

1. Slabs should be designed as continuous spans with negative bending reinforcing over the supports;
2. Additional reinforcing should be included to deter cracking caused by large temperature differences and to provide load distribution; and,
3. In areas where salt water; either brought into the structure by cars in winter or carried by the wind in coastal areas, may deteriorate the deck, protective measures must be taken. The top surface of the slab must be effectively sealed so that the salt water cannot migrate through the slab to the steel deck. A minimum G90 (Z275) galvanizing is recommended, and, the exposed bottom surface of the deck should be protected with a durable paint. The protective measures must be maintained for the life of the building.

4. Strong consideration should be given to using the steel deck as a stay in place form only, with the concrete slab then being designed to carry all loads as a reinforced concrete slab using reinforcing bars. Structural concrete design recommendations contained in ACI 215R and AASHTO-LRFD are suggested for guidance in the design of these slabs.

Additional information regarding steel floor deck in parking structures, including recommendations for concrete mix design and protection, may be found in AISC (2003)

12. Negative Reinforcement: When the slab is designed for negative moments, the deck shall be designed to act in the negative moment region only as a permanent form. Concrete in negative moment regions shall be designed by the designer as a conventional reinforced concrete slab in accordance with ACI 318. Design moments and shears shall be permitted to be calculated by any acceptable method of analysis which considers continuity. The coefficient method of ACI 318 Section 6.5 shall be considered to be an acceptable analysis method.

Commentary: Composite steel deck does not function as compression reinforcing steel in areas of negative moment. If the designer desires a continuous slab, then negative bending reinforcing should be designed using conventional reinforced concrete design techniques in compliance with ACI 318. The reinforcement chosen for temperature and shrinkage reinforcement most likely will not supply sufficient area of reinforcement for negative bending over the supports.

13. Cantilevered Slabs: At cantilevered slabs, the deck shall be considered to act only as a permanent form. The slab shall be designed by the designer for negative bending in accordance with ACI 318.

Commentary: At cantilevered slabs, the deck acts only as a permanent form. Composite steel deck does not function as compression reinforcing steel at cantilevers. Negative bending reinforcing at the cantilever should be designed using conventional reinforced concrete design techniques in compliance with ACI 318. The reinforcement chosen for temperature and shrinkage reinforcing most likely will not supply sufficient area of reinforcement for negative bending at the cantilever.

14. Diaphragm Shear Capacity: Diaphragm strength and stiffness shall be determined in accordance with AISI S310:

User Note: In instances where the required diaphragm capacity exceeds what can be calculated using AISI S310 a designer can potentially develop additional capacity by designing the diaphragm as a reinforced concrete diaphragm in accordance with ACI 318. This design option as a concrete diaphragm is outside the scope of this Standard.

User Note: SDI-DDM contains diaphragm load tables that comply with AISI S310.

15. Reinforcement for Temperature and Shrinkage:
    a. Reinforcement for crack control purposes other than to resist stresses from quantifiable structural loadings shall be permitted to be provided by one of the following methods:
        1. Welded wire reinforcement or reinforcing bars with a minimum area of 0.00075 times the area of the concrete above the deck (per foot or meter of width), but not be less than the area provided by 6 x 6 – W1.4 x W1.4 (152 x 152 – MW9 x MW9) welded wire reinforcement.
        2. Concrete specified in accordance with ASTM C1116, Type I, containing steel fibers meeting the criteria of ASTM A820, Type I , Type II, or Type V, at a dosage rate determined by the fiber manufacturer for the application, but not less than 25 lb/cu yd (14.8 kg/cu meter).
        3. Concrete specified in accordance with ASTM C1116, Type III, containing macrosynthetic fibers meeting the criteria of ASTM D7508 at a dosage rate determined by the fiber manufacturer for the application, but not less than 4 lb./cu yd (2.4 kg/m^3).
    b. When the slab is designed for negative moments in accordance with Section 2.4.B.12 or Section 2.4.B.13, temperature and shrinkage reinforcement shall be provided in accordance with Section 2.4.B.15.a. Section 7.6.4 of ACI 318 shall not apply.

User Note: It is suggested that if fibers are used for this purpose, that the designer include quality control provisions in accordance with ACI 544.3R in the project specifications.

Commentary: Concrete floor slabs employing Portland cement will start to experience a reduction in volume as soon as they are placed. Where shrinkage is restrained, cracking will occur in the floor. The use of the appropriate types and amount of reinforcement for shrinkage and temperature movement control is intended to result in a larger number of small cracks in lieu of a fewer number of larger cracks. Even with the best floor design and proper construction, it is unrealistic to expect crack free floors. Every owner should be advised by both the designer and contractor that it is normal to expect some amount of cracking and that such occurrence does not necessarily reflect adversely on either the adequacy of the floor's design or quality of the construction.

Cracking can be reduced when the causes are understood and preventative steps are taken in the design phase. The major factors that the designer can control concerning shrinkage and cracking include cement type, aggregate type and gradation, water content, water/cement ratio, and reinforcement.

Most measures that can be taken to reduce concrete shrinkage will also reduce the cracking tendency. Drying shrinkage can be reduced by using less water in the mixture and the largest practical maximum-size aggregate. A lower water content can be achieved by using a well-graded aggregate and lower initial temperature of the concrete. Designers are referred to ACI 302.1R and ACI 224.1 for additional information.

Although cracking is inevitable, properly placed reinforcement used in adequate amounts will reduce the width of individual cracks. By distributing the shrinkage strains, the cracks are distributed so that a larger number of narrow cracks occur instead of a few wide cracks. Additional consideration by the designer may be required to further limit the size and frequency of cracks. Additional provisions for crack control are frequently required where concrete is intended to be exposed, floors that will be subjected to wheel traffic, and floors which will receive an inflexible floor covering material (such as tile).

Modifications to fiber dosages will vary depending upon the specific fiber manufacturers' recommendations. As a general rule, reduced crack widths can be achieved by increasing the amount of steel reinforcement or by increasing the fiber dosage and/or minimizing the shrinkage potential of the concrete.

Because composite deck-slabs are typically designed as a series of simple spans, flexural cracks may form over supports. Flexural cracking of the concrete in negative moment regions of the slab (over beams and girders) is not typically objectionable unless the floor is to be left exposed or covered with inflexible floor coverings. Flexural cracking and crack widths can be minimized by one or more of the following: 1.) by paying strict attention to preventing overloads at deck midspan during construction, as this is a common source of flexural cracks; 2.) utilizing a stiffer steel deck; 3.) reducing the slab span. If flexural cracks must be strictly controlled, consideration should be given to designing the composite deck-slab for negative moments over supports (both beams and girders) and providing appropriate reinforcing steel at these supports.

16. Fire Resistance: The designer shall consider required fire resistance ratings in the design of the composite slab.

Commentary: Fire rating requirements may dictate the concrete strength or density. Many fire rated assemblies that use composite floor decks are available. In the Underwriters Laboratories *Fire Resistance Directory*, the composite deck constructions show hourly ratings for restrained and unrestrained assemblies. ASTM E119 provides information in Appendix X3 titled "Guide for Determining Conditions of Restraint for Floor and Roof Assemblies and for Individual Beams", indicating that deck attached to steel or concrete framing, and interior spans of wall supported deck may be considered to be restrained, while end spans of wall supported deck should be considered to be unrestrained. Designers should be aware that some fire rated assemblies set limits on load capacity and/or place restrictions on fastener type and spacing.

2.5 Accessories:

A. Accessories for structural applications shall be of dimensions and thickness suitable for the application, and shall be designed in accordance with AISI S100 or AISC 360, as applicable.

3. Execution

3.1 Installation/General:

A. Temporary shoring, if required, shall be designed to resist the loads indicated in Section 2.4.A.2. The shoring shall be designed and installed in accordance with standards applicable to the specific shoring system and shall be left in place until the concrete attains 75% of its specified design strength.

User Note: Typical practice is to retain shoring in place for a minimum of 7 days.

B. Deck Support Attachment: Steel deck shall be anchored to structural supports by arc spot welds, fillet welds, or mechanical fasteners. The average attachment spacing of deck at supports perpendicular to the span of the deck panel shall not exceed 16 inches (400 mm) on center, with the maximum attachment spacing not to exceed 18 inches (460 mm), unless more frequent fastener spacing is required for diaphragm design. The deck shall be adequately attached to the structure to prevent the deck from slipping off the supporting structure.

User Note: When the sidelap is a standing seam interlock, it may be permissible to only attach the female side, subject to design requirements, when the female hem holds the male leg down. When the sidelap is a nestable sidelap a single fastener through both sheets of steel deck is acceptable to secure both sheets.

C. Deck Sidelap Fastening: For deck with spans less than or equal to 5 feet (1.5 m), sidelap fasteners shall not be required. unless required for diaphragm design. For deck with spans greater than 5 feet (1.5 m), sidelaps shall be fastened at intervals not to exceed 36 inches (1 m) on center, unless more frequent fastener spacing is required for diaphragm design, using one of the following methods:

1. Screws with a minimum diameter of 0.190 inches (4.83 mm) (#10 diameter)
2. Crimp or button punch
3. Arc spot welds 5/8 inch (16 mm) minimum visible diameter, minimum 1-1/2 inch (38 mm) long fillet weld, or other weld shown to be substantially equivalent through testing in accordance with AISI S905, or by calculation in accordance with AISI S100, or other equivalent method approved by the authority having jurisdiction.
4. Other sidelap attachment methods approved by the authority having jurisdiction.

User Note: The above sidelap spacing is a minimum. Service loads or diaphragm design may require closer spacing or larger sidelap welds. Good metal-to-metal contact is necessary for a good sidelap weld. When welding, burn holes are to be expected and are not a grounds for rejection. The SDI does not recommend fillet welded or arc spot welded sidelaps for deck that is thinner than 0.0358 inch design thickness (20 gage) due to difficulty in welding thinner material.

D. Deck Perimeter Attachment Along Edges Between Supports: Support at the perimeter of the floor shall be designed and specified by the designer. For deck with spans less than or equal to 5 feet (1.5 m), perimeter attachment shall not be required, unless required for diaphragm design. For deck with spans greater than 5 feet (1.5 m), perimeter edges of deck panels between span supports shall be fastened to supports at intervals not to exceed 36 inches (1 m) on center, unless more frequent fastener spacing is required for diaphragm design, using one of the following methods:

1. Screws with a minimum diameter of 0.190 inches (4.83 mm) (#10 diameter)
2. Arc spot welds with a minimum 5/8 inch (16 mm) minimum visible diameter, or minimum 1-1/2 inch (38 mm) long fillet weld.
3. Powder actuated or pneumatically driven fasteners.

User Note: This condition is often referred to as parallel attachment to supports, referring to the support members running parallel or nearly parallel with the flutes of the deck panel. Number 10 screws may not be adequate at thicker edge supports and may fracture due to driving torque resistance. A minimum of a Number 12 screw is recommended at parallel edge supports thicker than 14 gage (0.0747 inch) and a Number14 screw may be required for thicker and harder steels.

E. Cantilevers:

1. Sidelaps shall be attached at the end of the cantilever and at a maximum spacing of 12 inches (300 mm) on center from the cantilevered end at each support.
2. Each deck corrugation shall be fastened at both the perimeter support and the first interior support.
3. The deck shall be completely attached to the supports and at the sidelaps before any load is applied to the cantilever.
4. Concrete shall not be placed on the cantilever before concrete is placed on the adjacent span.

F. Minimum fastener edge distances shall be determined in accordance with AISI S100.

G. Deck bearing surfaces shall be brought into contact as required by the fastening method.

Commentary: Out of plane support flanges can create knife-edge supports and air gaps between the deck and support. This makes welding more difficult and allows distortion under screw or power actuated fastener washers or heads. Inherent tolerances of the supporting structure should be considered.

3.2 Welding

A. All welding of deck shall be in accordance with AWS D1.3. Each welder shall demonstrate the ability to produce satisfactory welds using a procedure in accordance with AWS D1.3.

User Note: SDI-MOC describes a weld quality control test procedure that can be used as a preliminary check for welding machine settings under ambient conditions.

B. For connection of the deck to the supporting structure, weld washers shall be used with arc spot welds on all deck units with metal thickness less than 0.028 inches (22 gage) (0.71 mm). Weld washers shall be a minimum thickness of 0.050 inches (1.27 mm) and have a nominal 3/8 inch (10 mm) diameter hole. Weld washers shall not be used between supports along the sidelaps.

User Note: AWS D1.3 and AISI S100 do not recommend the use of weld washers for welding sheet steel over 0.028 inches (22 gage) in thickness.

C. Where weld washers are not required, a minimum visible 5/8 inch (16 mm) diameter arc spot weld or arc seam weld of equal perimeter shall be used. Weld metal shall penetrate all layers of deck material at end laps and shall have good fusion to the supporting members.

D. When used, fillet welds to support structure shall be at least 1-1/2 inches (38 mm) long.

E. When steel headed stud anchors are installed to develop composite action between the beam or joist and the concrete slab, the steel headed stud anchor shall be permitted as a substitute for an arc spot weld to the supporting structure, subject to minimum fastener edge distance requirements for arc spot welds in accordance with AISI S100. Steel headed stud anchors shall be installed in accordance with AWS D1.1.

F. Deck bearing surfaces to be welded shall be brought into contact as required by AWS D1.3, Section 5.3.2.

3.3 Mechanical Fasteners

A. When the support fasteners are powder actuated or pneumatically driven, the acceptable range of support thickness, fastener spacing limitations, and the strength per fastener shall be based on the manufacturers' applicable fastener test report or other documentation acceptable to the designer and authority having jurisdiction.

B. Screws shall be acceptable for use without restriction on structural support thickness, however, the screw selected shall have a grip range compatible with the combined thickness of the deck and supporting member.

User Note: Mechanical fasteners (screws, powder or pneumatically driven fasteners, etc.) are recognized as viable anchoring methods, provided the type and spacing of the fastener satisfies the design criteria. Documentation in the form of test data, design calculations, or design charts should be submitted by the fastener manufacturer as the basis for obtaining approval. Strength of mechanically fastened connections are dependant upon both deck and support thickness.

3.4 Accessory Attachment:

A. Structural accessories shall be attached to supporting structure or deck as required for transfer of forces, but at a spacing not to exceed 12 inches (300 mm) on center. Non-structural accessories shall be attached to supporting structure or deck as required for serviceability, but spaced not to exceed 24 inches (600 mm) on center.

B. Mechanical fasteners or welds shall be permitted for accessory attachment.

3.5 Cleaning Prior to Concrete Placement:

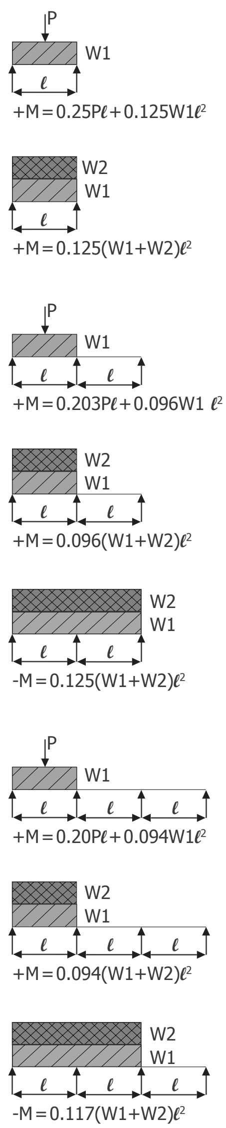
A. Surfaces shall be cleaned of debris, including but not limited to, welding rods, stud ferrules which are broken free from the stud, and excess fasteners, prior to concrete placement.

3.6 Reinforcing steel shall be installed when required by the construction documents.

Appendix 1

Composite Deck Construction Loading Diagrams

FIGURE 1
Loading Diagrams and Bending Moments



Simple Span Condition

$+M = 0.25P\ell + 0.125W1\ell^2$

$+M = 0.125(W1+W2)\ell^2$

Double Span Condition

$+M = 0.203P\ell + 0.096W1\ \ell^2$

$+M = 0.096(W1+W2)\ell^2$

$-M = 0.125(W1+W2)\ell^2$

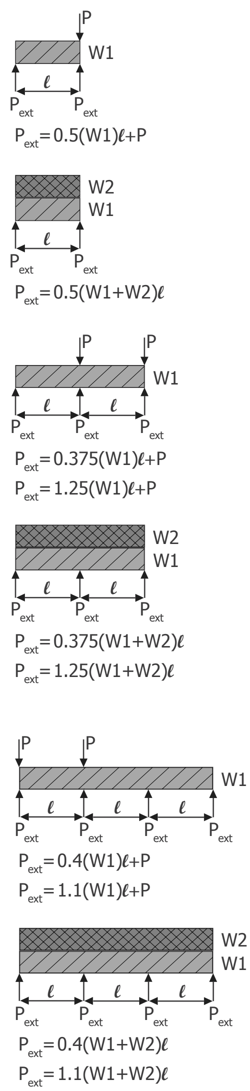
Triple Span Condition

$+M = 0.20P\ell + 0.094W1\ell^2$

$+M = 0.094(W1+W2)\ell^2$

$-M = 0.117(W1+W2)\ell^2$

FIGURE 2
Loading Diagrams and Support Reactions



Simple Span Condition

$P_{ext} = 0.5(W1)\ell + P$

$P_{ext} = 0.5(W1+W2)\ell$

Double Span Condition

$P_{ext} = 0.375(W1)\ell + P$

$P_{ext} = 1.25(W1)\ell + P$

$P_{ext} = 0.375(W1+W2)\ell$

$P_{ext} = 1.25(W1+W2)\ell$

Triple Span Condition

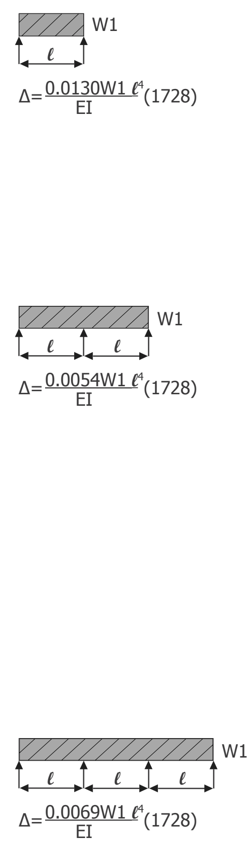
$P_{ext} = 0.4(W1)\ell + P$

$P_{ext} = 1.1(W1)\ell + P$

$P_{ext} = 0.4(W1+W2)\ell$

$P_{ext} = 1.1(W1+W2)\ell$

FIGURE 3
Loading Diagrams and Deflections



Simple Span Condition

$\Delta = \frac{0.0130W1\ \ell^4}{EI}(1728)$

Double Span Condition

$\Delta = \frac{0.0054W1\ \ell^4}{EI}(1728)$

Triple Span Condition

$\Delta = \frac{0.0069W1\ \ell^4}{EI}(1728)$

User Note:

See section 1.3 for requirement for construction loads to be shown on construction documents and section 2.4.A for construction loads.

Notes for Figures 1, 2, and 3

P	=	concentrated construction live load
I	=	in^4/ft. - deck moment of inertia
W1	=	slab weight + deck weight
W2	=	uniform construction live load
E	=	29.5 x 10^6 psi
ℓ	=	clear span length (ft.)

Dimensional consistency requires consistent units when calculating deflections.

Appendix 2

Strength Determination of Composite Deck-Slab by Pre-qualified Section Method

A2.1 General

1. This Appendix provides methods for the calculation of strength of composite steel deck-slabs. It shall be permitted to use this method if steel headed stud anchors (studs) are not present on the beam flange supporting the composite steel deck, or if steel headed stud anchors are present in any quantity.
2. Limitations:
   A. Deck shall be limited to galvanized or uncoated steel decks with embossments meeting the requirements for Type I, Type II, or Type III patterns as shown in Figure A2.1, A2.2, A2.3, and A2.4. The design embossment height, p_h, shall not be less than 0.035 in (0.89 mm) and shall not be greater than 0.105 in (2.67 mm). Embossments shall not be less than 90% of the design embossment depth.
   B. The embossment factor, p_s, shall not be less than that defined in Table A2-1.

Table A2-1 Minimum Embossment Factor

Deck Embossment Type	Nominal Deck Depth	Minimum p_s
1	1.5 in	5.5
1	2.0 in	12.0
1	3.0 in	18.0
2	1.5 in	5.5
2	2.0 in	8.5
2	3.0 in	8.5
3	1.5 in	5.5
3	2.0 in	10.0
3	3.0 in	12.0

a. For Type 1 deck embossments:

$$p_{s1} = 12\,(l_e / S) \qquad \text{(Eq. A2-1)}$$

b. For Type 2 deck embossments:

$$p_{s2} = 12\,(\,l_1 + l_2\,) / S \qquad \text{(Eq. A2-2)}$$

c. For Type 3 deck embossments:

$$p_{s1} = 12\,(\text{sum of } l_1 \text{ lengths within } S_1) / S_1 \qquad \text{(Eq. A2-3)}$$

$$p_{s2} = 12\,(\text{sum of } l_2 \text{ lengths within } S_2) / S_2 \qquad \text{(Eq. A2-4)}$$

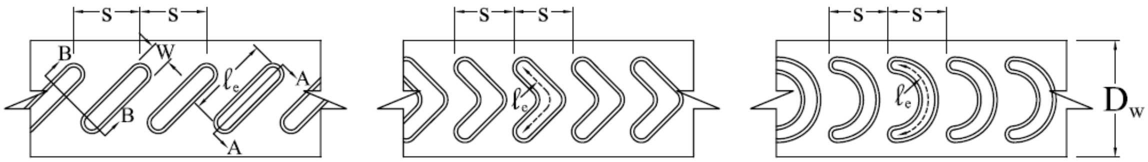


Figure A2.1 – Type 1 Embossments with length measured along centerline

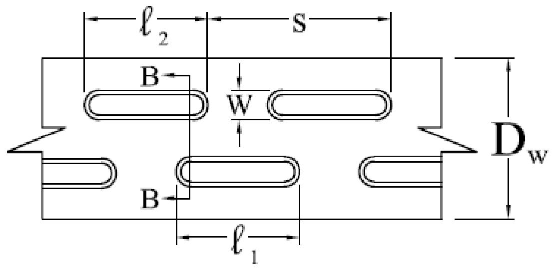


Figure A2.2 – Type 2 Embossments

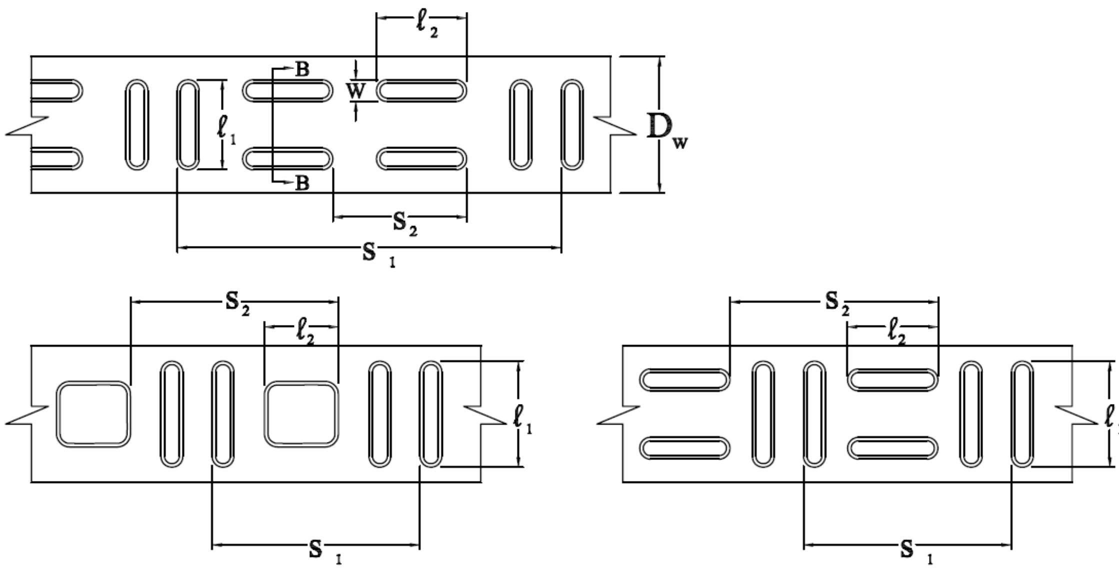


Figure A2.3 – Type 3 Embossments

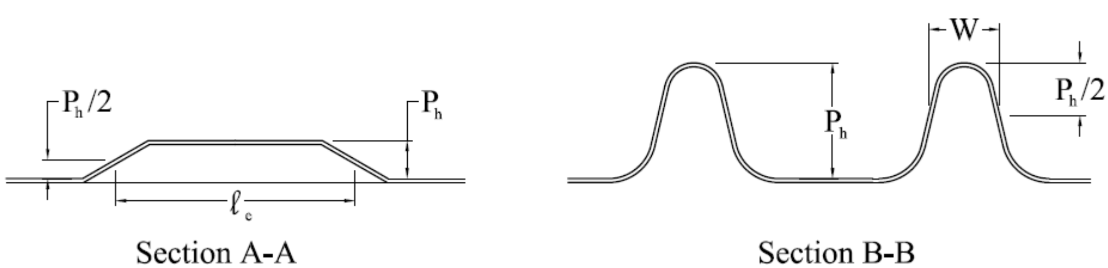


Figure A2.4 – Embossment Section Details

C. The web angle measured from the horizontal plane, θ, shall be limited to values between 55° and 90° and the webs shall have no reentrant bends in their flat width.

D. The deck section depth, d_d, shall be less than or equal to 3 in. (75 mm)

E. All sheet steel used for deck shall comply with Section 2.1 of this Standard.

F. Concrete shall comply with Section 2.1 of this Standard.

G. The concrete thickness above the steel deck shall be equal to or greater than 2 inches (50mm).

H. Composite deck-slabs shall be classified as under-reinforced or over-reinforced.

a. Slabs with (c/d) less than the balanced condition ratio $(c/d)_b$ shall be considered under-reinforced, whereas slabs with (c/d) greater than or equal to $(c/d)_b$ shall be considered over-reinforced. The compression depth ratio shall be calculated as:

$$(c/d) = \frac{A_s F_y}{0.85 f'_c d b \beta_1} \qquad \text{(Eq. A2-5)}$$

The compression depth ratio for the balanced condition shall be calculated as:

$$(c/d)_b = \frac{0.003(h - d_d)}{\left(F_y/E_s + 0.003\right)d} \qquad \text{(Eq. A2-6)}$$

Where:

A_s	=	area of steel deck, in^2/ft (mm^2/m) of slab width
b	=	unit width of compression face of composite slab, 12 in.(1000 mm)
c	=	distance from extreme compression fiber to composite neutral axis, in. (mm)
d	=	distance from extreme compression fiber to centroid of steel deck, in. (mm)
d_d	=	overall depth of steel deck profile, in. (mm)
E_s	=	modulus of elasticity of steel deck, psi (MPa)
f'_c	=	specified compressive strength of concrete, psi (MPa)
F_y	=	specified yield strength of steel deck, psi (MPa)
h	=	nominal out-to-out depth of slab, in. (mm)
β_1	=	0.85 if $f'_c \le 4000$ psi (27.58 MPa)

$$\beta_1 = 1.05 - 0.05\left(\frac{f'_c}{1000}\right) \geq 0.65 \text{ if } f'_c > 4000 \text{ psi}$$
(Eq. A2-7a) (in-lb)

$$\beta_1 = 1.09 - 0.008f'_c \geq 0.65 \text{ if } f'_c > 27.58 \text{ MPa}$$
(Eq. A2-7b) (SI)

3. The strength of a composite deck-slab shall be the least of the following strength limit states:
   A. Flexural strength
   B. One-way shear strength in accordance with Section 2.4.B.7
4. For load combinations that include concentrated loads, punching shear in accordance with Section 2.4.B.8 shall be considered.
5. Cracked section properties shall be determined by Appendix 5.

A2.2 Flexural Strength: This section shall be used to determine the flexural strength of the composite deck-slab.

1. Under-Reinforced: The nominal moment capacity shall be calculated as follows:

   A. The resisting moment, M_{no}, of the composite section shall be determined based on cracked section properties.

   M_{no} = $K\,M_y$ (Eq. A2-8)

   Where:
   M_y = Yield moment for the composite deck-slab, considering a cracked cross section
   = $F_y I_{cr} / (h-y_{cc})$ (Eq A2-9)

   K = $(K_3/K_1) \leq 1.0$ (Eq. A2-10)
   F_y = yield stress of steel deck, psi (MPa)
   h = slab depth measured from top of concrete to bottom of deck, in (mm)
   I_{cr} = cracked section moment of inertia, in^4 (mm^4)
   M_{no} = nominal resisting moment, kip-in (N-mm)
   y_{cc} = distance from top of slab to neutral axis of cracked section, in (mm)
   K_1, K_3 = Coefficients of deck profile and embossment pattern

   B. K_3 shall be calculated as follows

   K_3 = 1.4

User Note: Using $K_3 = 1.4$ is appropriate for typical deck applications where the floor is multiple deck panels wide. For instances where the floor is relatively narrow, measured perpendicular to the deck span, the following equation may yield more accurate (conservative) results.

$$K_3 = 0.87 + 0.0688N - 0.00222N^2 \leq 1.4 \quad \text{(Eq. A2-11)}$$

C. K_1 shall be calculated as follows

For Type 1 Embossment Deck Panels:

$$K_1 = 0.07\,(D_W)^{0.5} / p_h \leq 1.55 \quad \text{(Eq. A2-12)}$$

For Type 2 Embossment Deck Panels:

$$K_1 = 15\,(t) / \left[D_W\,(p_h)^{0.5}\right] \quad \text{(Eq. A2-13)}$$

For Type 3 Embossment Deck Panels:

$$K_1 = \frac{(K_{11})p_{s1} + (K_{12})p_{s2}}{p_{s1} + p_{s2}} \quad \text{(Eq. A2-14)}$$

Where:

D_w = Width of flat portion of the deck web, in.
K_{11} = K_1 calculated for Type 1 embossments in Type 3 pattern
K_{12} = K_1 calculated for Type 2 embossments in Type 3 pattern
N = Number of cells in a slab width
= w / R
w = Slab width, in.(mm)
R = Repeating pattern or cell spacing, in.(mm)
p_h = Embossment height, in.
t = Deck thickness, in.

D. The flexural resistance (ASD) of the composite section shall be calculated as follows.

$$M_{no} / \Omega_s = K\,M_y / \Omega \quad \text{(Eq. A2-15)}$$

$$\Omega_s = 1.75$$

The flexural resistance (LRFD) of the composite section shall be calculated as follows.

$$\Phi_s M_{no} = \Phi_s K M_y \qquad \text{(Eq. A2-16)}$$

$$\Phi_s = 0.85$$

2. Over-reinforced Slabs $(c/d) \geq (c/d)_b$

A. The flexural resistance, in positive bending, of an over-reinforced composite slab shall be determined by:

LRFD

$$M_{ro} = \phi_c f_c' b \beta_1 (d - \beta_1 c/2) \leq \Phi_s K M_y \qquad \text{(Eq. A3-6a)}$$

ASD

$$M_{ro} = f_c' b \beta_1 (d - \beta_1 c/2)/\Omega_c \leq K M_y/\Omega_s \qquad \text{(Eq. A3-6b)}$$

Where:

$$c = d\left\{\sqrt{\rho m + \left(\frac{\rho m}{2}\right)^2} - \frac{\rho m}{2}\right\} \qquad \text{(Eq. A3-7)}$$

$$\rho = \frac{A_s}{bd} \qquad \text{(Eq. A3-8)}$$

$$m = \frac{E_s \varepsilon_{cu}}{f_c' \beta_1} \qquad \text{(Eq. A3-9)}$$

E_s = modulus of elasticity of steel deck
= 29,500,000 psi (203,000 MPa)
K = As calculated for an underreinforced slab
ε_{cu} = 0.003
ϕ_c = 0.65
Ω_c = 2.30
Φ_s = 0.85
Ω_s = 1.75

B. Equation A3-6 is valid only for composite slabs where no part of the steel deck has yielded.

Appendix 3

Strength Determination of Composite Deck-Slab by Shear Bond Method

A3.1 General

1. This Appendix provides methods for the calculation of strength of composite steel deck-slabs by the shear bond method. It shall be permitted to use this method if steel headed stud anchors (studs) are not present on the beam flange supporting the composite steel deck, or if steel headed stud anchors are present in any quantity.
2. Limitations:
   A. Deck shall be limited to galvanized or top surface uncoated steel decks.
   B. All sheet steel used for deck shall comply with Section 2.1 of this Standard.
   C. Concrete shall comply with Section 2.1 of this Standard.
   D. The concrete thickness above the steel deck shall be equal to or greater than 2 inches (50mm).
3. The strength of a composite deck-slab shall be the least of the following strength limit states:
   A. Shear bond resistance
   B. Flexural strength
   C. One-way shear strength in accordance with Section 2.4.B.7
4. For load combinations that include concentrated loads, punching shear in accordance with Section 2.4.B.8 shall be considered.

A3.2 Shear bond Resistance

1. The ultimate shear bond resistance of a composite slab section shall be calculated using parameters determined from a testing program of full-scale slab specimens in accordance with SDI-T-CD. The shear bond resistance (V_r) of a composite slab shall be determined as follows:

LRFD

$$V_r = \phi_v V_t \qquad \text{(Eq. A3-1a)}$$

ASD

$$V_r = V_t / \Omega_v \qquad \text{(Eq. A3-1b)}$$

Where,

V_r = shear bond resistance, pounds/ft (N/m) of slab width,
V_t = tested shear bond resistance, pounds/ft (N/m) of slab width, determined in accordance with SDI-T-CD,
ϕ_v = 0.75
Ω_v = 2.00

2. The permissible uniform load for shear bond shall be:

$$W_r = 2V_r/L \quad \text{(Eq. A3-2)}$$

Where:
L = deck design span, ft. (m)

A3.3 Flexural Strength

1. Composite slabs subject to flexural failure shall be classified as under-reinforced or over-reinforced slabs depending on the compression depth ratio, (c/d). Slabs with (c/d) less than the balanced condition ratio $(c/d)_b$ shall be considered under-reinforced, whereas slabs with (c/d) greater than or equal to $(c/d)_b$ shall be considered over-reinforced. The compression depth ratio shall be calculated as:

$$(c/d) = \frac{A_s F_y}{0.85 f'_c d b \beta_1} \quad \text{(Eq. A3-3)}$$

The compression depth ratio for the balanced condition shall be calculated as:

$$(c/d)_b = \frac{0.003(h - d_d)}{\left(F_y/E_s + 0.003\right)d} \quad \text{(Eq. A3-4)}$$

Where:

A_s = area of steel deck, in^2/ft (mm^2/m) of slab width
b = unit width of compression face of composite slab, 12 in.(1000 mm)
c = distance from extreme compression fiber to composite neutral axis, in. (mm)
d = distance from extreme compression fiber to centroid of steel deck, in. (mm)
d_d = overall depth of steel deck profile, in. (mm)
E_s = modulus of elasticity of steel deck
= 29,500,000 psi (203,000 MPa)
f'_c = specified compressive strength of concrete, psi (MPa)
F_y = specified yield strength of steel deck, psi (MPa)
h = nominal out-to-out depth of slab, in. (mm)
β_1 = 0.85 if $f'_c \leq 4000$ psi (27.58 MPa)

$$\beta_1 = 1.05 - 0.05\left(\frac{f'_c}{1000}\right) \geq 0.65 \text{ if } f'_c > 4000 \text{ psi}$$

$$\beta_1 = 1.09 - 0.008 f'_c \geq 0.65 \text{ if } f'_c > 27.58 \text{ MPa}$$

2. Under-reinforced Slabs $(c/d) < (c/d)_b$

A. The moment resistance, in positive bending, of an under-reinforced composite slab shall be taken as:

LRFD

$M_r = \phi_s M_y$ (Eq. A3-5a)

ASD

$M_r = M_y / \Omega_s$ (Eq. A3-5b)

M_y = Yield moment for the composite deck-slab, considering a cracked cross section

$= F_y I_{cr} / (h - y_{cc})$

Where:

ϕ_s = 0.85

Ω_s = 1.75

F_y = yield stress of steel deck, psi (MPa)

I_{cr} = cracked section moment of inertia, in^4 (mm^4)

h = slab depth, in (mm)

y_{cc} = distance from top of slab to neutral axis of cracked section, in (mm)

3. Over-reinforced Slabs $(c/d) \geq (c/d)_b$

A. The moment resistance, in positive bending, of an over-reinforced composite slab shall be determined by:

LRFD

$$M_{ro} = \phi_c f'_c b \beta_1 c \left(d - \beta_1 c/2\right) \leq \Phi_s M_y \quad \text{(Eq. A3-6a)}$$

ASD

$$M_{ro} = f'_c b \beta_1 c \left(d - \beta_1 c/2\right)/\Omega_c \leq M_y / \Omega_s \quad \text{(Eq. A3-6b)}$$

Where:

$$c = d\left\{\sqrt{\rho m + \left(\frac{\rho m}{2}\right)^2} - \frac{\rho m}{2}\right\} \quad \text{(Eq. A3-7)}$$

$$\rho = \frac{A_s}{bd} \quad \text{(Eq. A3-8)}$$

$$m = \frac{E_s \varepsilon_{cu}}{f'_c \beta_1} \quad \text{(Eq. A3-9)}$$

E_s	=	modulus of elasticity of steel deck
	=	29,500,000 psi (203,000 MPa)
ε_{cu}	=	0.003
ϕ_c	=	0.65
Ω_c	=	2.30
ϕ_s	=	0.85
Ω_s	=	1.75

B. Equation A3-6 is valid only for composite slabs where no part of the steel deck has yielded.

Appendix 4

Strength Determination of Composite Deck-Slab by Ultimate Strength Method

A4.1 General

1. This Appendix provides a method for the calculation of strength of composite steel deck-slabs by the ultimate strength method. It shall be permitted to use this method if steel headed stud anchors (studs) are present on the beam flange supporting the composite steel deck.
2. Limitations:
   A. Deck shall be limited to galvanized or top surface uncoated steel decks.
   B. All sheet steel used for deck shall comply with Section 2.1 of this Standard.
   C. Concrete shall comply with Section 2.1 of this Standard.
   D. The concrete thickness above the steel deck shall be equal to or greater than 2 inches (50mm).
3. The deck shall be capable of developing composite action with the concrete slab by mechanical means.

> **User Note:** Mechanical means of developing composite action with the concrete slab include, but are not limited to:
> (a) indentations or embossments on the deck web, flange, or both.
> (b) transverse wires or bars welded to the top flange of the deck.
> (c) reentrant angle webs, referred to as keystone or dovetail profiles.
> Deck which develops composite action solely by chemical bond without mechanical bond does not meet this requirement.

4. The strength of a composite deck slab shall be the least of the following strength resistance limit states:
   A. Flexural strength as calculated by this Appendix.
   B. One-way shear strength in accordance with Section 2.4.B.7
5. For load combinations that include concentrated loads, punching shear in accordance with Section 2.4.B.8 shall be considered.
6. Cracked section properties shall be determined in accordance with Appendix 5.
7. The term “stud” shall refer to a steel headed stud anchor.
8. Mechanical fasteners, such as screws or power-actuated fasteners, or welds shall not be permitted to substitute for studs for purposes of calculation of flexural strength using this method.
9. Studs shall be either 3/4 inch (19 mm) or 7/8 inch (22 mm) in diameter.
10. Composite deck-slabs shall be classified as under-reinforced or over-reinforced.

a. Slabs with (c/d) less than the balanced condition ratio $(c/d)_b$ shall be considered under-reinforced, whereas slabs with (c/d) greater than or equal to $(c/d)_b$ shall be considered over-reinforced. The compression depth ratio shall be calculated as:

$$(c/d) = \frac{A_s F_y}{0.85 f_c' d b \beta_1} \qquad \text{(Eq. A4-1)}$$

The compression depth ratio for the balanced condition shall be calculated as:

$$(c/d)_b = \frac{0.003(h - d_d)}{\left(F_y / E_s + 0.003\right)d} \qquad \text{(Eq. A4-2)}$$

Where:

A_s = area of steel deck, in^2/ft (mm^2/m) of slab width

b = unit width of compression face of composite slab, 12 in.(1000 mm)

c = distance from extreme compression fiber to composite neutral axis, in. (mm)

d = distance from extreme compression fiber to centroid of steel deck, in. (mm)

d_d = overall depth of steel deck profile, in. (mm)

E_s = modulus of elasticity of steel deck
= 29500 ksi (203,000 MPa)

f_c' = specified compressive strength of concrete, psi (MPa)

F_y = specified yield strength of steel deck, psi (MPa)

h = nominal out-to-out depth of slab, in. (mm)

β_1 = 0.85 if $f_c' \leq 4000$ psi (27.58 MPa)

$$\beta_1 = 1.05 - 0.05\left(\frac{f_c'}{1000}\right) \geq 0.65 \text{ if } f_c' > 4000 \text{ psi}$$

(Eq. A4-3a) (in-lb)

$$\beta_1 = 1.09 - 0.008 f_c' \geq 0.65 \text{ if } f_c' > 27.58 \text{ MPa}$$

(Eq. A4-3b) (SI)

A2.2 Flexural Strength: This section shall be used to determine the flexural strength of the composite deck-slab.

1. Under-Reinforced: The nominal moment capacity shall be calculated as follows:

   a. The nominal (ultimate) moment capacity with studs on beam shall be calculated as follows:

$$M_{nu} = A_s F_y (d - a/2) \quad \text{(Eq. A4-4)}$$

Where:

A_s = steel deck cross sectional area per unit width of steel deck

a = developed depth of concrete in the compression zone

$$= \frac{A_s F_y}{0.85 f'_c b} \quad \text{(Eq. A4-5)}$$

b = unit width

d = distance from the top of the slab to the centroid of the steel deck

f'_c = concrete strength, ksi

F_y = yield stress of steel deck

M_{nu} = nominal (ultimate) moment capacity with studs on beam

   b. The minimum number of studs per unit width required to develop the ultimate moment capacity, M_{nu}, shall be calculated as follows:

N_{su} = minimum number of studs per unit width required to develop the ultimate moment capacity of the composite deck-slab

$$= F_T / Q_n \quad \text{(Eq. A4-6)}$$

Where:

A_{bf} = deck bottom flange area per unit width of steel deck

A_{webs} = deck web area per unit width of steel deck

A_{sa} = cross sectional area of steel headed stud anchor

F_T = required anchorage force per unit deck width to develop the full cross section of the steel deck

$$= F_y \left(A_s - \frac{A_{webs}}{2} - A_{bf} \right) \quad \text{(Eq. A4-7)}$$

Q_n = nominal shear strength of steel headed stud anchor

$$= 0.5 A_{sa} \sqrt{f'_c E_c} \quad \text{(Eq. A4-8)}$$

c. When the number of studs per unit width, N_s, installed equals or exceeds N_{su}, then:

$$M_n = M_{nu} \quad \text{(Eq. A4-9)}$$

d. When the number of studs per unit width, N_s, installed is less than N_{su}, then:

$$M_n = M_{no} + (M_{nu}-M_{no})(N_s/ N_{su}) \quad \text{(Eq. A4-10)}$$

Where:

M_{no} = nominal moment capacity of the deck-slab without studs, calculated by either Appendix 2 or Appendix 3 or by testing in accordance with SDI T-CD
N_s = number of studs installed per unit width

e. The flexural resistance shall be taken as:

LRFD
$$M_r = \phi_s M_n \quad \text{(Eq. A4-11a)}$$

ASD
$$M_r = M_n / \Omega_s \quad \text{(Eq. A4-11b)}$$

Where:
ϕ_s = 0.85
Ω_s = 1.75

2. Over-reinforced $(c/d) \geq (c/d)_b$

a. The moment resistance, in positive bending, of an over-reinforced composite slab shall be determined by:

LRFD
$$M_{ro} = \phi_c f_c' b \beta_1 (d - \beta_1 c/2) \leq \Phi_s M_y \quad \text{(Eq. A4-12a)}$$

ASD
$$M_{ro} = f_c' b \beta_1 (d - \beta_1 c/2)/\Omega_c \leq M_y / \Omega_s \quad \text{(Eq. A4-12b)}$$

Where:

$$c = d\left\{\sqrt{\rho m + \left(\frac{\rho m}{2}\right)^2} - \frac{\rho m}{2}\right\} \qquad \text{(Eq. A4-13)}$$

$$\rho = \frac{A_s}{bd} \qquad \text{(Eq. A4-14)}$$

$$m = \frac{E_s \varepsilon_{cu}}{f_c' \beta_1} \qquad \text{(Eq. A4-15)}$$

E_s = 29,500,000 psi (203000 MPa)
ε_{cu} = 0.003
ϕ_c = 0.65
Ω_c = 2.30
ϕ_s = 0.85
Ω_s = 1.75

b. Equation A4-12 is valid only for composite slabs where no part of the steel deck has yielded.

A4.3 Placement of Studs

1. Studs shall not be installed greater than 36 inches (914 mm) on center.
2. At butted end laps, the studs shall be installed through the deck ends on both sides of the butt joint. Studs shall be in sufficient quantity, N_s, on both sides of the joint to develop the required strength.
3. At perimeter conditions or openings (where slabs are discontinuous) the studs shall be welded through the deck to engage the deck end.

User Note: Figures A4.1, A4.2, and A4.3 illustrate stud installation.

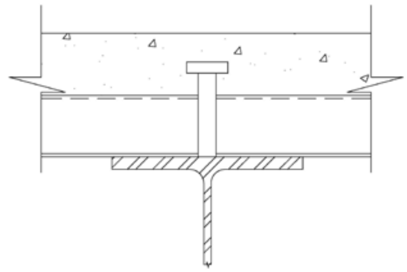


Figure A4.1 - Studs installed not at a deck end or butt joint are installed through the deck

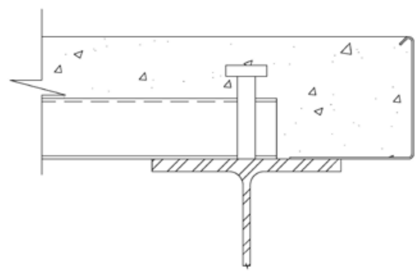


Figure A4.2 - Studs at a deck end are required to anchor the deck and must be installed through the deck

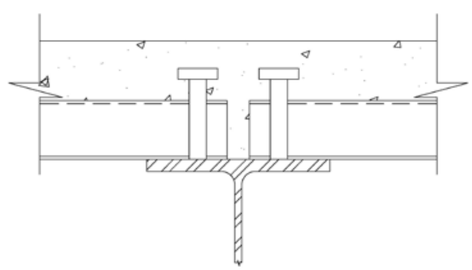


Figure A4.3 - Studs at a deck butt joint must be installed individually through deck ends and not centered on the butt joint. The full number of studs anchoring the deck, Ns, must be installed on each end of the deck at both sides of the butt joint.

Appendix 5

Section Properties of Composite Deck-Slabs

A5.1 General

This Appendix provides methods for the calculation of geometric cross section properties for composite steel deck cross sections with concrete. Alternate methods of rational analysis which consider material properties and cracked cross section properties shall be permitted.

User Note: This method will provide conservative results for slabs with reinforcing. The designer may choose to use alternate methods that consider the contribution of the reinforcing steel in this case.

A5.2 Transformed Composite Neutral Axis

The distance y_{cc} from the extreme compression fiber of the concrete to the neutral axis of the transformed composite section shall be determined from Figure A5-1 and Equations A5-1 and A5-3.

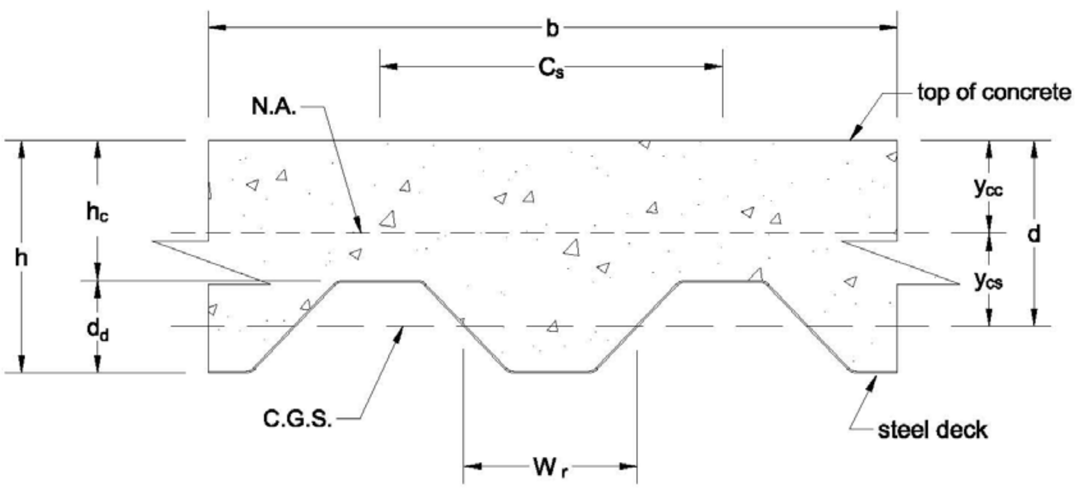


Figure A5-1 – Composite Section

Note: Section shows non-cellular deck. Section shall be either cellular, a blend of cellular and non-cellular, or non-cellular deck. Unless testing is performed that demonstrates that the interlocking device is capable of developing the full strength of the cross-section, only the element in contact with the concrete shall be considered in the design.

C.G.S. = centroidal neutral axis of full, unreduced cross section of steel deck, in. (mm)

C_s = pitch of deck ribs in. (mm)

N.A.	=	neutral axis of transformed composite section
W_r	=	average deck rib width in. (mm)
d_d	=	depth of deck in. (mm)
h	=	overall slab depth in. (mm)
h_c	=	depth of concrete above steel deck in. (mm)

A5.3 Moment of Inertia of the Cracked Section

For the cracked moment of inertia

$$y_{cc} = d\left[\sqrt{2\rho n + (\rho n)^2} - \rho n\right] \leq h_c \qquad \text{(Eq. A5-1)}$$

where

ρ	=	$\dfrac{A_s}{bd}$
A_s	=	area of steel deck per unit slab width in^2. (mm^2)
b	=	unit slab width (12 inches in imperial units)
d	=	distance from top of concrete to centroid of steel deck
n	=	modular ratio = $\dfrac{E_s}{E_c}$
E_s	=	modulus of elasticity of steel deck
	=	29500 ksi (203,000 MPa)

E_c	=	modulus of elasticity of concrete	
	=	$w_c^{1.5}\,(f'_c)^{0.5}$,ksi ; or	f'_c in ksi
	=	$w_c^{1.5}\,33(f'_c)^{0.5}$,psi ; or	f'_c in psi
	=	$57000\,(f'_c)^{0.5}$,psi ; or	f'_c in psi
	=	$0.043 w_c^{1.5}\,(f'_c)^{0.5}$,MPa ; or	f'_c in kg/m^3
	=	$4700\,(f'_c)^{0.5}$,MPa	f'_c in kg/m^3

w_c	=	concrete unit weight, pcf (kg/m^3)
f'_c	=	concrete strength, ksi or psi (MPa)

$y_{cs} = d - y_{cc}$ where y_{cc} shall be determined from Equation A5-1.

The cracked moment of inertia transformed to steel, I_c ,shall be calculated using Equation A5-2.

$$I_c = \frac{b}{3n} y_{cc}^3 + A_s y_{cs}^2 + I_{sf} \qquad \text{(Eq. A5-2)}$$

where

I_{sf}	=	moment of inertia of the full (unreduced) steel deck per unit slab width. in^4. (mm^4)

A5.4 Moment of Inertia of the Uncracked Section

For the uncracked moment of inertia

$$y_{cc} = \frac{0.5bh_c^2 + nA_s d + W_r d_d \left(h - 0.5d_d\right)\frac{b}{C_s}}{bh_c + nA_s + W_r d_d \frac{b}{C_s}} \qquad \text{(Eq. A5-3)}$$

The uncracked moment of inertia transformed to steel, I_u ,shall be calculated using Equation A5-4.

$y_{cs} = d - y_{cc}$ where y_{cc} shall be determined from Equation A5-3.

$$I_u = \frac{bh_c^3}{12n} + \frac{bh_c}{n}\left(y_{cc} - 0.5h_c\right)^2 + I_{sf} + A_s y_{cs}^2 + \frac{W_r bd_d}{nC_s}\left[\frac{d_d^2}{12} + \left(h - y_{cc} - 0.5d_d\right)^2\right]$$

(Eq. A5-4)

A5.5 Moment of Inertia of the Composite Section

The moment of inertia of the composite section considered effective for deflection computations shall be calculated by Equation A5-5.

$$I_d = \frac{I_u + I_c}{2} \qquad \text{(Eq. A5-5)}$$