STEEL DECK INSTITUTE sdi ®

AMERICAN NATIONAL STANDARDS INSTITUTE/ STEEL DECK INSTITUTE

SD - 2022 Standard for Steel Deck

Approved American National Standard ANSI

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Since hazards may be associated with the handling, installation, or use of steel deck and its accessories, prudent construction practices should always be followed. The Steel Deck Institute recommends that parties involved in the handling, installation or use of steel deck and its accessories review all applicable manufacturers' material safety data sheets, applicable rules and regulations of the Occupational Safety and Health Administration and other government agencies having jurisdiction over such handling, installation or use, and other relevant construction practice publications.

Preface

(This Preface is not part of the ANSI/SDI SD-2022, *Standard for Steel Deck*, but is included for informational purposes only.)

This Specification is based upon past successful usage, advances in the state of knowledge, and changes in design practice. The Steel Deck Institute *Standard for Steel Deck*, ANSI/SDI SD-2022, provides an integrated treatment of steel *roof deck*, *non-composite steel floor deck*, and *composite steel floor deck*, and replaces earlier SDI C, NC, and RD Standards. Designs can be made according to either ASD or LRFD provisions.

This Standard has been developed as a consensus document to provide a uniform practice in the design of *steel deck* used for roof and floor applications. The intention is to provide design criteria for routine use and not to provide specific criteria for infrequently encountered problems, which occur in the full range of structural design.

The Symbols and Appendices to this Standard are an integral part of the Standard. A non-mandatory Commentary has been prepared to provide background for the Standard provisions and the user is encouraged to consult it. Additionally, non-mandatory User Notes are interspersed throughout the Standard to provide concise and practical guidance in the application of the provisions.

The user is cautioned that professional judgment must be exercised when data or recommendations in the Standard are applied, as described more fully in the disclaimer notice preceding this Preface.

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Symbols

Some definitions in the list below have been simplified in the interest of brevity. In all cases, the definitions given in the body of the Standard govern. Symbols without text definitions, or used only in one location and defined at that location, are omitted in some cases. The section or table number in the right-hand column refers to the Section where the symbol is first used.

Symbol	Definition	Section
A_{bf}	*Deck* bottom flange area per unit width of *steel deck*	F3.2.3
A_c	Concrete area available to resist shear, in.^{2} (mm^2), defined as the concrete contained within the webs of the *deck* and the projection of the webs into the slab above the *deck*	F4
A_s	*Steel deck* cross sectional area per unit width of *steel deck*	F3.2.3
A_{sa}	Cross sectional area of steel headed stud anchor	F3.2.3
A_{webs}	*Deck* web area per unit width of *steel deck*	F3.2.3
D_w	Width of flat portion of the *deck* web, in.	F3.2.1
E_s	Modulus of elasticity of *steel deck* (29,500 ksi (203,000 MPa))	F2.1
F_T	Required anchorage force per unit *deck* width to develop the full cross section of the *steel deck*	F3.2.3
F_u	Specified tensile strength of sheet steel	F3.1
F_{us}	Specified minimum tensile strength of a steel headed stud anchor	F3.2.3
F_y	Specified yield stress of sheet steel	A3.1
I_{cr}	Cracked section moment of inertia of *composite steel deck-slab*, in.^{4} (mm^4)	F3.2
K	*Embossment* effectiveness factor	F3.2
K_1, K_3	Coefficients of *deck* profile and *embossment* pattern	F3.2
K_{11}	K_1 calculated for Type 1 *embossments* in Type 3 pattern	F3.2
K_{12}	K_1 calculated for Type 2 *embossments* in Type 3 pattern	F3.2
L	*Deck* design span	F3.2
M_{no}	Nominal resisting moment of *composite steel deck-slab*, kip-in (N-mm)	F3.2
M_{nu}	Nominal (ultimate) moment capacity of *composite steel deck-slab* with studs on beam	F3.2.3
M_y	Yield moment for the *composite steel deck-slab*, considering a cracked cross section	F3.2.1
N	Number of cells in a slab width	F3.2.1
N_s	Number of studs installed per unit width	F3.2.3

N_{su}	Minimum number of studs per unit width required to develop the ultimate moment capacity of the *composite steel deck-slab*	F3.2.3
P_{lc}	Concentrated construction live load per unit width of *deck* section	App. 2.1.1
Q_n	Nominal shear strength of steel headed stud anchor	F3.2.3
R	Repeating pattern or cell spacing, in.(mm)	F3.2.1
R_a	Required strength for ASD	A4.1
R_n	Nominal resistance	A4.1
R_u	Required strength for LRFD	A4.1
S, S_1, S_2	*Deck embossment* spacing	F3.2.1
V_c	Nominal one-way shear resistance of concrete	F4
V_D	Nominal shear strength of the *steel deck* section calculated in accordance with AISI S100, kips (kN)	F4
V_{pr}	Punching shear resistance, kips (kN)	F5
W_n	Nominal uniform load for shear bond	F3.2.2
a	Developed depth of concrete in the compression zone	F3.2.3
b_o	Perimeter of critical section, in. (mm)	F5
d	Distance from extreme compression fiber of *composite steel deck-slab* to centroid of *steel deck*, in. (mm)	F3.2.3
d_d	Overall depth of *steel deck* profile, in. (mm)	F3.2.1
f'_c	Specified compressive strength of concrete, psi (MPa)	B1
h_c	Thickness of concrete cover above *steel deck*, in. (mm)	F3.2.1
ℓ_e, ℓ_1, ℓ_2	*Deck embossment* length	F3.2.1
p_h	Design *embossment* height	F3.2.1
p_s, p_{s1}, p_{s2}	*Deck embossment* factor	F3.2.1
w_{dc}	Dead weight of fluid concrete	App. 2.1.1
w_{dd}	Dead weight of the *steel deck*	App. 2.1.1
w_{lc}	Uniform construction live load (combined with fluid concrete)	App. 2.1.1
w_{cdl}	Uniform construction live load (combined with bare *deck*)	App. 2.1.1
w_{lcc}	Uniform construction live load applied to cantilever span and adjacent span	App. 2.2.2
β_c	Ratio of long side to short side of concentrated load or reaction area	F5
λ	Concrete density adjustment factor	F4
ϕ	Resistance factor (LRFD)	A4.1
Ω	Safety factor (ASD)	A4.1

SECTION A - General Provisions

A1 Scope, Applicability, and Units

A1.1 Scope

This Standard for Steel Deck, hereafter referred to as the Standard, shall govern the materials, design, and erection of cold-formed *steel deck* used in floor and roof applications in buildings and similar structures. It shall be applicable to applications where the *steel deck* is used as the only load carrying element, where the *steel deck* acts as a temporary or permanent form for structural or non-structural concrete or other cementitious materials, and for applications where the *steel deck* acts as a permanent form and provides positive flexural reinforcement for a *composite steel deck-slab*. This Standard governs the design of the resulting *composite steel deck-slab*.

The Appendices shall be part of the Standard.

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.

A1.2 Units of Symbols and Terms

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.

A2 Reference Codes, Standards, and Documents

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 318-19, Building Code Requirements for Structural Concrete
2. American Institute of Steel Construction (AISC)
   a. ANSI/AISC 360-22, Specification for Structural Steel Buildings
3. American Iron and Steel Institute (AISI)
   a. AISI S100-16 w/S2-20 (2020), North American Specification for the Design of Cold-Formed Steel Structural Members
   b. AISI S310-20, North American Standard for the Design of Profiled Steel Diaphragm Panels

c. AISI S905-17, Test Methods for Mechanically Fastened Cold-Formed Steel Connections

4. American Society of Civil Engineers (ASCE)
   a. ASCE/SEI 7-22, Minimum Design Loads and Associated Criteria for Buildings and Other Structures
   b. ASCE/SEI 8-21, Specification for the Design of Cold-Formed Stainless Steel Structural Members
5. American Society for Testing and Materials (ASTM)
   a. ASTM A924 / A924M - 17a, Standard Specification for General Requirements for Steel Sheet, Metallic-Coated by the Hot-Dip Process
6. American Welding Society (AWS)
   a. AWS D1.1:2015, Structural Welding Code-Steel
   b. AWS D1.3:2018, Structural Welding Code-Sheet Steel
7. Steel Deck Institute (SDI)
   a. SDI T-CD-2022, Test Standard for Composite Steel Deck-Slabs
8. Steel Joist Institute (SJI)
   a. ANSI/SJI 200-2015, Standard Specification for CJ-Series Composite Steel Joists

A3 Material

A3.1 Applicable Steels

A3.1.1 Sheet Steel for Deck and Accessories that Carry Defined Loads

Sheet steel for *deck* and *accessories* that carry defined loads shall conform to AISI S100, Section A3.

a. All sheet steel used for this purpose shall have a minimum specified yield stress that meets or exceeds 33 ksi (230 MPa).

b. 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.

c. For the case where the *steel deck* acts as tensile reinforcement for the *composite steel deck-slab*, the steel shall conform to the following limits on the design yield stress for *composite steel deck-slab* behavior:

   (1) 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.

   (2) 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.

A3.1.2 Sheet Steel for Non-Structural Accessories

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.

A3.1.3 Deck Finish

Galvanizing or other metallic coatings shall conform to the requirements of the applicable steels in AISI S100, Section A3.

A3.2 Concrete and Reinforcement

1. Concrete placed on *steel deck* shall conform to ACI 318, Chapter 19, except as modified by Sections A3.2.2, A3.2.3, and A3.2.4 of this Standard.
2. Except for *composite steel deck-slabs*, the specified concrete compressive strength for structural concrete slabs shall not be less than that permitted by ACI 318 nor the applicable building code.
3. For *composite steel deck-slabs*, 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 steel deck-slab* shall not exceed 6000 psi (42 MPa), unless strength and flexural stiffness of slabs with stronger concrete has been justified by testing.
4. Admixtures containing chloride salts or other substances that are corrosive or otherwise deleterious to the *steel deck* and imbedded items shall not be permitted.
5. Steel reinforcing shall conform to ACI 318, Section 20.2.

A4 Design Basis

A4.1 Required Strength

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

Where the available strength [factored resistance] is calculated by this Standard, it shall satisfy the following equations:

For ASD,

$$R_a \leq R_n / \Omega \qquad \text{(Eq. A4.1.1)}$$

Where:

R_a = Required strength for ASD
R_n = Nominal resistance
Ω = Safety factor

For LRFD,

$$R_u \leq \phi\, R_n \qquad \text{(Eq. A4.1.2)}$$

Where:

R_u = Required strength for LRFD
ϕ = Resistance factor

A4.2 Section Properties and Deck Strength

The section properties and allowable strength (ASD) or design strength (LRFD) for the *steel deck* shall be computed in accordance with AISI S100.

1. *Deck* which is perforated in the web or flange or both shall be permitted to be designed either by testing in accordance with Section K of AISI S100, or by methods of rational analysis.

A4.3 Structural Accessories

Accessories for structural applications shall be designed in accordance with AISI S100 or AISC 360, as applicable.

A4.4 Minimum Bearing Length

Minimum bearing length shall be determined in accordance with AISI S100.

A5 Specification of Sheet Steel Thickness

It shall be permitted to specify base sheet 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.

A6 Definitions

Terms which are not defined in this Standard, but are defined in AISI S100, shall have the meaning as defined in AISI S100. Terms not defined in this Standard nor AISI S100 shall have the ordinary accepted meaning for the context for which they are intended.

Accessory**:** Cold-formed steel components of the *steel deck* system other than the *steel deck*, which may include, but are not limited to; *gage* metal *pour stops*, girder fillers, ridge, hip and valley plates, end closures and *sump pans*.

Authority Having Jurisdiction (AHJ)**:** Organization, political subdivision, office or individual charged with the responsibility of administering and enforcing the provisions of the applicable building code.

Butt Joint**:** The condition in which the ends of adjacent *steel deck panels* are placed closely together over a supporting member, but not overlapped.

Button Punch**:** A mechanical means of connecting two adjacent pieces of *steel deck* together at a *side-lap* condition by crimping with a special tool. This process requires a specific *steel deck side-lap* profile.

Clenched Connection**:** A mechanical means of connecting two pieces of sheet metal together by punching through the steel to create a flap of metal which is then crimped. This is done with a proprietary tool on the seams or *side-laps* of interlocking *steel deck panels*.

Composite Steel Floor Deck**:** A specific *steel deck* profile used as a form to create a structural concrete slab with the *steel deck* as moment reinforcement. The *steel deck* has *embossments*, interlocking profile geometry, or other horizontal shear resistance devices to develop mechanical bond between the *steel deck* and concrete so the slab compositely resists vertical and diaphragm shear loads. Prior to composite action, the *steel deck* acts as a *form deck* and work platform.

Composite Steel Deck-Slab**:** A system comprised of structural concrete placed over *composite steel floor deck*, in which the *steel deck* acts as positive bending reinforcing for the slab during the service life of the structure.

Corrugation**:** The *pitch*, depth, folds and bends that create a distinctive profiled *steel deck panel* shape.

Deck**:** (See *Steel Deck*)

Designer**:** The licensed professional responsible for the content of the drawings and *specifications* from which the *steel deck* is to be constructed.

User Note: The *Designer* is usually the structural engineer-of-record, however it may be the architect or other licensed professional acting within the scope of their license.

Embossments**:** Regularly spaced indentations, or lugs, on the various surfaces of a *steel deck* profile for the purpose of achieving composite action between the profiled sheet and the cured structural concrete.

End Lap**:** The end-to-end overlap of adjacent *steel deck panels* perpendicular to the *steel deck panel* fluting. The *steel deck panels* must be of the same profile to properly overlap the ends. Some *steel deck* profiles have shapes that are not capable of overlapping.

Fastener Pattern**:** The frequency and spacing of fasteners at each support and *side-lap* for proper securement of *steel deck panels* to the structure.

Fastening**:** The act of securing the *steel deck* in a combination of fastener, fastened material and *fastener pattern* in the final position.

Finish**:** In *deck* terminology, the coating on the *steel deck* surface such as unpainted, painted, galvanized or galvanized and painted.

Floor Deck**:** *Steel deck* used in composite or non-composite floor construction.

Flute**:** A fabricated fold or bend in a *steel deck panel* which projects downward from a horizontal plane to form a repetitive groove or undulation and is comprised of one *corrugation*.

Form Deck: (See *Non-Composite Steel Floor Deck*)

Gage: A measure of thickness for sheet steel.

Interlocking Side-Lap or Interlocking Seam: *Steel deck panels* having male and female side edges. Adjacent *steel deck panels* have a male and female edge which interlock with one another at the time of installation. The *interlocking seams* or *side-laps* are usually raised above the supports and are fastened together using *button punches*, proprietary punch systems, welds, or screws.

Nestable Side-Lap or Lapped Seam: A *steel deck panel* that contains a partial *rib* on one side which overlaps, or “nests" into the side edge of the adjacent *steel deck panel*, which contains a full *rib*. The nesting usually occurs at the lowest point of the *deck* profile.

Non-Composite Steel Floor Deck: *Steel deck* used as a stay-in-place form for structural concrete. May be designed to resist *superimposed loads* in a non-composite manner. (See *Form Deck*)

Panel: A single piece of *steel deck*.

Pitch: The center-to-center distance between the repeating *corrugation* patterns of the *steel deck panel*.

Pour Stop: A steel angle, or bent plate, used around the perimeter of a floor or opening to contain the concrete during placement.

Rib: A fabricated fold or bend in a *steel deck panel* which projects upward from a horizontal plane and is comprised of one *corrugation*.

Roof Deck: *Steel deck panels* used in a structural manner as a base for constructing and supporting the roof insulation and membrane.

Side-Lap Fastener: A weld, screw, *button punch*, or crimp, not penetrating a support, used to connect the sides of two adjacent sheets of *steel deck* together. It may also be called a stitch fastener.

Side-Lap: The lap at the longitudinal edges of adjacent *steel deck panels* which are attached by self-drilling screws, welds, *button punches*, or crimps.

Specifications: The portion of the contract documents that consists of the written requirements for materials, standards and workmanship.

Standing Seam: A type of raised joint between adjacent *steel deck panels* made by turning up the edges of two adjacent *steel deck panels* and then folding, or interlocking, them in a variety of ways.

Steel Deck: Cold-formed steel *panels* installed on support framing in a roof or floor, including steel *roof deck*, *non-composite steel floor deck*, and *composite steel floor deck*. May also be referred to as metal *deck*, *decking* or just *deck*.

***Stiffener*:** A formed groove, bead projection or depression usually parallel to the longitudinal axis of the *steel deck* used to strengthen the flat element against local buckling or to minimize oil canning.

***Stitch Screws*:** Fasteners used to secure the lap at the longitudinal edges (*side-laps*) of *steel deck panels* between supporting members.

***Substrate*:** Members or components which support the weight of *steel deck* and any applied loads.

***Superimposed Load*:** Load carried by the *deck* or deck-slab other than the dead load of the *deck* and concrete (if any).

***Support Connection*:** A fastener or weld attaching one or more sheets to supporting members. Also called a structural *steel deck* connection.

***Sump Pan*:** A *gage* metal *roof deck accessory* comprised of a flat sheet or recessed basin used to aid in the support of roof drain assemblies.

SECTION B - Construction Documents

B1 Construction Documents

The construction documents shall describe the *steel decks* and *steel deck* supported concrete slabs that are to be constructed and shall include not less than the following information:

1. Loads
   a. Floor or roof loads as required by the applicable building code
   b. Concentrated loads (where applicable)
   c. Assumed construction phase loads
2. Structural framing plans for all *deck* areas showing the span direction and the size, location and type of all *deck* supports
3. Deck and Deck Attachment
   a. Depth, type (profile), and design thickness
   b. *Deck* material (including yield strength) and *deck finish*
   c. *Deck* attachment type, spacing, and details
   d. Any special coatings on the *deck*, factory or field applied (if required)
4. Concrete and Reinforcing (where required)
   a. Specified concrete strength, f'_c
   b. Specified concrete density (and tolerance if required for fire rating assembly)
   c. Specified strength or grade of reinforcing steel or welded wire reinforcement
   d. Size, extent and location of all reinforcement
   e. Slab thicknesses (and tolerance if required for fire rating assembly)
   f. Discontinuous fiber reinforcement material, type and dosage (if used in *composite steel deck-slab*)
   g. Specify if the concrete is to be placed to a uniform thickness, or to a level surface
5. Temporary shoring requirements (if required for *deck* supporting fluid concrete)
6. Fire rating (if required), including assembly designation and *deck finish* requirements

User Note: Information to aid the *Designer* in selecting the proper *deck finish* is provided in the Commentary.

SECTION C - Design of Steel Deck for Construction Phase Loads

C1 Deck Carrying Wet Concrete

The *steel deck* section alone shall have adequate strength and stiffness to support the construction phase loads applied prior to the concrete attaining 75% of its specified strength, f'_c in accordance with Appendix 2. When shored, the effect of the shoring on the span length shall be included in the evaluation.

User Note: This is applicable to *roof deck* or *floor deck* (composite or non-composite) carrying a cast-in-place concrete slab.

C2 Deck Not Carrying Wet Concrete

The *steel deck* section shall have adequate strength and stiffness to support all loads applied during construction. Loads and load combinations in accordance with Appendix 2 shall be considered.

User Note: This section is applicable to *roof deck* or *floor deck* (non-composite) not carrying a cast-in-place concrete slab. This section would include *floor deck* supporting a thin gypsum concrete topping, or a cementitious floor *panel*. This would also include *roof deck* carrying a lightweight insulating concrete topping.

SECTION D - Design of Steel Deck for In-Service Conditions – General Provisions

D1 Strength Design

Deck shall be evaluated for strength using the loads and load combinations required by the applicable building code. In the absence of a building code, the loads and load combinations prescribed by ASCE 7 shall be used.

D2 Fire Resistance

The *Designer* shall consider required fire resistance ratings in the design of the *deck* and slab (if any).

D3 Deck Deflection

Floor and *roof deck* deflection under service load shall be limited by the provisions of the applicable building code and by the requirements of any applied materials.

User Note: In absence of a building code, recommended roof deflection limits are provided in the Commentary.

SECTION E - Design of Steel Deck with Cementitious Topping without Composite Deck Action for In-Service Conditions

User Note: This section refers to *non-composite (form) deck* and is equally applicable to *deck* on composite or non-composite supporting steel beams, open web steel joists, cold-formed steel framing, or other support framing.

In addition to the requirements of Section D, the following shall apply.

E1 Structural Concrete Slab Design

The concrete slab shall be designed as a one-way or two-way slab by the *Designer* in accordance with ACI 318.

E1.1 Cantilevers

At cantilever slabs and regions of negative moment, negative bending reinforcement shall be designed and provided, unless the *deck* is capable of resisting all loads and the concrete is considered to be a wearing surface only.

E1.2 Temperature and Shrinkage

Temperature and shrinkage effects in the concrete shall be as required by ACI 318 for one-way or two-way slabs. The *Designer* shall be permitted to consider only the area of concrete above the *deck* when designing for temperature and shrinkage.

E1.3 Concrete Thickness

The concrete thickness above the top of the *steel deck* shall not be less than 1-1/2 inches (38 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.

User Note: The 1-1/2 inch minimum concrete thickness is applicable when the *deck* is on non-composite steel framing. When using *steel deck* on composite steel beams, AISC 360, Section I3.2 requires the following: (1) a minimum slab thickness above the *steel deck* of 2 inches, (2) steel headed stud anchors extend a minimum of 1-1/2 inches above the top of the *steel deck*, and (3) specified concrete cover is a minimum of 1/2 inch above the top of the steel headed stud anchor.

E1.4 Calculation of Slab Superimposed Load Capacity

1. When Temporary Shoring is Not Used

   The weight of the concrete slab shall be permitted to be supported by the non-composite deck, and not be deducted from the nominal strength of the structural concrete slab when calculating the available strength under *superimposed loads*.

EXCEPTION: Unless protective measures are used, when the non-composite deck is permanently exposed to an environment where it can reasonably be considered to be prone to deterioration, the weight of the slab shall be deducted from the nominal strength of the structural concrete slab when calculating the available strength under *superimposed loads*.

2. When Temporary Shoring is Used

The effect of shoring removal shall be considered when determining the available strength of the concrete slab. Conservatively, the weight of the concrete slab shall be permitted to be deducted from the nominal strength of the structural concrete slab when calculating the available strength under *superimposed loads*.

EXCEPTION: Unless protective measures are used, when the non-composite deck is permanently exposed to an environment where it can reasonably be considered to be prone to deterioration, the weight of the slab shall be deducted from the nominal strength of the structural concrete slab when calculating the available strength under *superimposed loads*.

User Note: Refer to the Commentary for discussion of when *deck* may be prone to deterioration.

E2 Design of Deck to Carry Dead and Live Loads

It shall be permitted to design the *deck* to carry all required dead and live loads, without consideration of contribution from a structural or non-structural topping material.

EXCEPTION: Unless protective measures are used, when the non-composite deck is permanently exposed to an environment where it can reasonably be considered to be prone to deterioration, the *deck* shall not be permitted to carry in-service loads.

SECTION F - Design of Composite Steel Deck-Slabs for In-Service Conditions

In addition to the requirements of Section D, the following shall apply.

F1 General

F1.1 Load Determination

The *superimposed load* capacity shall be determined by deducting the weight of the slab and the *deck* from the total load capacity. The effect of the removal of temporary shoring shall be considered for *composite steel deck-slabs* governed by *deck* to concrete bond strength (Section F3.2).

EXCEPTION: Unless protective measures are used, when the *composite steel floor deck* is permanently exposed to an environment where it can reasonably be considered to be prone to deterioration, *composite steel floor deck* action shall not be assumed.

User Note: Refer to the Commentary for discussion of when *deck* may be prone to deterioration.

Unless *composite steel deck-slabs* are designed for continuity, slabs shall be assumed to act on simple spans.

F1.2 Concrete

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. Concrete shall comply with Section A3.2.3.

F1.3 Steel and Deck Finish

All sheet steel used for *deck* shall comply with Section A3.1.1.c of this Standard. The surface of the *deck* in contact with the concrete shall not be coated (except for zinc galvanizing in accordance with ASTM A924 (A924M)) unless the shear bond strength has been established by testing in accordance with SDI T-CD.

F2 Properties of the Steel Deck-Slab Section

F2.1 General Assumptions

Properties of *composite steel deck-slabs* shall be determined by methods of rational engineering mechanics, based on the following assumptions:

1. Strain in the *steel deck* (and any reinforcing steel, if considered) shall be assumed directly proportional from the distance from the neutral axis.
2. Maximum usable strain at the extreme concrete compression fiber shall be assumed equal to 0.003.
3. Stress in *steel deck* (and any reinforcing steel, if considered) shall be taken as E_s times the steel strain, not to exceed F_y.
4. The tensile strength of the concrete shall be neglected.
5. The relationship between concrete compressive stress distribution and concrete strain shall be as permitted by ACI 318, Section 22.2.2.3.

F2.2 Reinforcement Classification

Where required by methods of design for flexure, *composite steel deck-slabs* shall be classified as under-reinforced, balanced, or over-reinforced by methods of rational engineering mechanics.

User Note: The Commentary provides a method of determining this classification using generally accepted methods.

F2.3 Cracked Section Properties

Cracked section properties shall be determined by methods of rational engineering mechanics.

User Note: The Commentary provides a method for determining these properties using generally accepted methods.

F3 Flexural Resistance of Composite Deck-Slabs

The flexural resistance of a *composite steel deck-slab* shall be the minimum of the ultimate flexural strength (Section F3.1) and the flexural strength governed by *deck* to concrete bond (Section F3.2).

F3.1 Ultimate flexural strength

The ultimate flexural strength shall be calculated by Section F3.1.1

The following safety factors, Ω (ASD) and resistance factors, ϕ (LRFD) shall be applied to the nominal moment capacity calculated for the *composite steel deck-slab*, for the applicable limit state(s):

Limit State	Ω (ASD)	ϕ (LRFD)
Flexure (under-reinforced when $F_u/F_y > 1.08$)	1.75	0.85
Flexure (under-reinforced when $F_u/F_y \le 1.08$)	2.30	0.65
Flexure (over-reinforced)	2.30	0.65

F3.1.1 General Strain Analysis

The general strain analysis method shall be based upon rational engineering analysis considering the general assumptions in Section F2.1.

The general strain analysis method is permitted to be used for determining the nominal moment capacity of *composite steel deck-slabs* with or without additional reinforcing steel, and for *composite steel deck-slabs* that are under-reinforced or over-reinforced.

F3.2 Flexural strength controlled by deck-to-concrete bond

The flexural strength controlled by *deck*-to-concrete bond shall be calculated by either Section F3.2.1, Section F3.2.2, or Section F3.2.3.

F3.2.1 Prequalified Section Method

This Section provides methods for the calculation of the flexural strength of under-reinforced *composite steel deck-slabs* using the "Prequalified Section Method."

F3.2.1.1 Limitations

1. *Deck* shall have *embossments* meeting the requirements for Type 1, Type 2, or Type 3 patterns as shown in Figures F3.2-1, F3.2-2, F3.2-3, and F3.2-4. 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* are permitted to project into or away from the concrete.
2. The *deck embossment* factor, p_s, shall not be less than that defined in Table F3.2-1

Table F3.2-1 Minimum Deck Embossment Factor

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

a. For Type 1 *deck embossments*:

$$p_{s1} = 12\,(\ell_e / S) \qquad \text{(Eq. F3.2-1)}$$

b. For Type 2 *deck embossments*:

$$p_{s2} = 12\,(\ell_1 + \ell_2) / S \qquad \text{(Eq. F3.2-2)}$$

c. For Type 3 *deck embossments*:

$$p_{s1} = 12\,(\text{sum of } \ell_1 \text{ lengths within } S_1) / S_1 \qquad \text{(Eq. F3.2-3)}$$

$$p_{s2} = 12\,(\text{sum of } \ell_2 \text{ lengths within } S_2) / S_2 \qquad \text{(Eq. F3.2-4)}$$

$$p_{s1} + p_{s2} \geq p_s \qquad \text{(Eq. F3.2-5)}$$

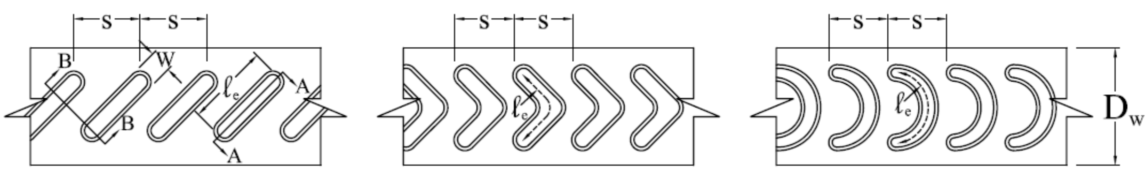


Figure F3.2-1 – Type 1 Embossments with length measured along centerline

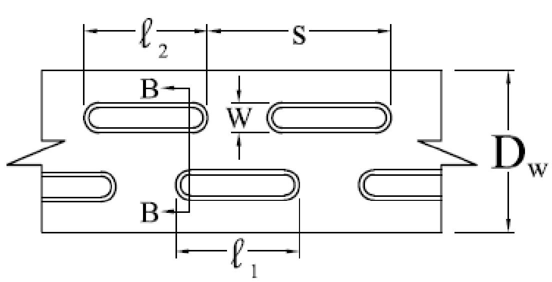


Figure F3.2-2 – Type 2 Embossments

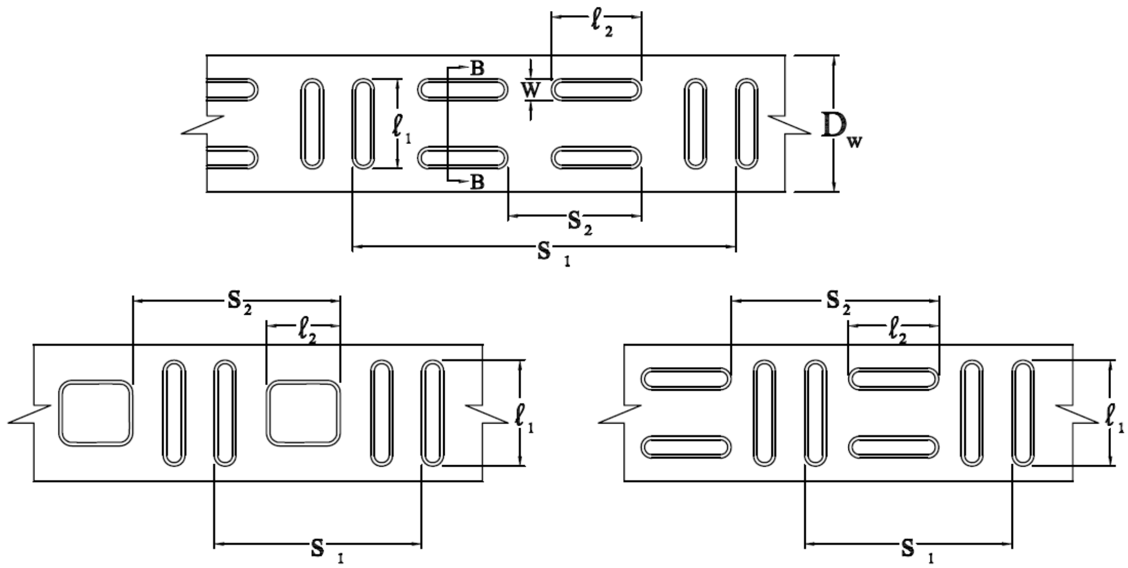


Figure F3.2-3 – Type 3 Embossments

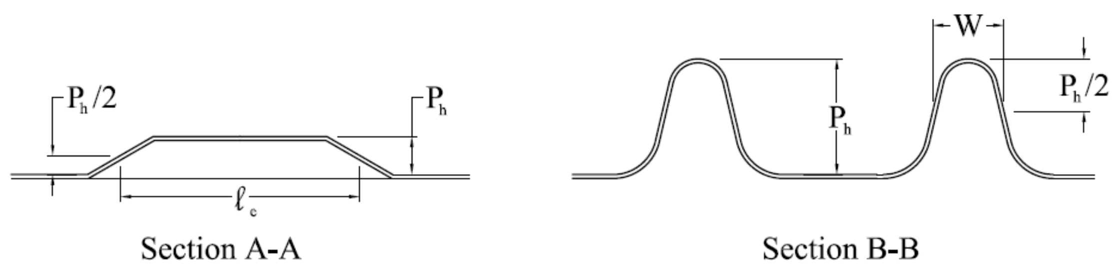


Figure F3.2-4 – Embossment Section Details

3. 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.
4. The *deck* section depth, d_d, shall not be less than 1.5 in. (37 mm) nor greater than 3 in. (75 mm).
5. The *rib pitch* shall not be less than 6 in. (152 mm) nor greater than 12 in. (305 mm).
6. The base metal thickness, t, shall not be less than 0.028 in. (0.71 mm) nor greater than 0.060 in. (1.52 mm).
7. The concrete thickness above the *deck*, h_c, shall not exceed 6 in. (152 mm) for the purposes of calculating yield moment, M_y.

User Note: The limitation of 6 in. concrete thickness above the *steel deck*, h_c, is only a limitation of calculating M_y and does not prohibit greater depths of concrete cover for purposes other than calculating M_y.

F3.2.1.2 Embossment Effectiveness Factor

The *embossment* effectiveness factor, K, shall be calculated as follows:

K = $(K_3/K_1) \le 1.0$ (Eq. F3.2-6)
K_1, K_3 = Coefficients of *deck* profile and *embossment* pattern
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.

$$K_3 = 0.87 + 0.0688N - 0.00222N^2 \le 1.4$$

K_1 shall be calculated as follows:

For Type 1 *embossment deck panels*:

$K_1 = 0.07\,(D_W)^{0.5} / p_h \le 1.55$ (Eq. F3.2-7)

For Type 2 *embossment deck panels*:

$K_1 = 15\,(t) / [D_W\,(p_h)^{0.5}]$ (Eq. F3.2-8)

For Type 3 *embossment deck panels*:

$K_1 = [(K_{11})\,p_{s1} + (K_{12})\,p_{s2}] / (p_{s1} + p_{s2})$ (Eq. F3.2-9)

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* base steel thickness, in.

F3.2.1.3 Flexural Strength

The nominal moment capacity shall be calculated as follows:

a. The resisting moment, M_n, of the composite section shall be determined based on cracked section properties.

$M_n = K\,M_y \leq (87.5\,\alpha^2)\,L^2$ (Eq. F3.2-10)

The nominal shear from any applied loading, V_n, shall not exceed $(350\,\alpha)L^2$.

Where:

L	=	*deck* design span, ft. (m)
M_n	=	nominal resisting moment of *composite steel deck-slab*, kip-in. (N-mm)
M_y	=	Yield moment for the *composite steel deck-slab*, considering a cracked cross section
	=	$F_y\,I_{cr} / (h - y_{cc})$ (Eq. F3.2-11)
F_y	=	yield stress of *steel deck*, psi (MPa)
I_{cr}	=	cracked section moment of inertia, in.^{4} (mm^4)
h	=	nominal out-to-out slab depth, in. (mm)
y_{cc}	=	distance from top of slab to neutral axis of cracked section, in. (mm)
α	=	1 if L is in feet; 0.3048 if L is in meters

The following safety factors, Ω (ASD) and resistance factors, ϕ (LRFD) shall be applied to the nominal moment capacity calculated for the *composite steel deck-slab*, for the applicable limit state(s):

Limit State	Ω (ASD)	ϕ (LRFD)
Flexure (under-reinforced)	1.75	0.85

F3.2.2 Shear Bond Method

F3.2.2.1 General

This Section provides methods for the calculation of strength of *composite steel deck-slabs* by the shear bond method.

The following safety factors, Ω (ASD) and resistance factors, ϕ (LRFD) shall be applied to the nominal moment capacity calculated for the *composite steel deck-slab*, for the applicable limit state(s):

Limit State	Ω (ASD)	ϕ (LRFD)
Shear Bond	Calculated in accordance with SDI T-CD	Calculated in accordance with SDI T-CD

F3.2.2.2 Shear bond Resistance

1. The 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 nominal strength determined from testing, R_n, shall be used to determine the nominal flexural resistance of the *composite steel deck-slab*.

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

   $$W_n = 2R_n / L \quad \text{(Eq. F3.2-12)}$$

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

   Nominal loads for other than uniform loading shall be determined by rational engineering analysis, based on the shear between supports and points of maximum shear on the beam shear diagram.

3. The nominal resisting moment for a *composite steel deck-slab* carrying a uniform load shall be:

   $$M_n = V_t L / 4 = W_n L^2/8 \quad \text{(Eq. F3.2-13)}$$

   Nominal resisting moments for other than uniform loading shall be determined by rational engineering analysis.

F3.2.3 End Anchorage by Steel Headed Stud Anchors Welded Through Deck

F3.2.3.1 General

1. This Section provides a method for the calculation of the nominal flexural resistance of under-reinforced *composite steel deck-slabs* by anchorage of the *deck*. It shall be permitted to use this method if steel headed stud anchors (studs) are present on the beam flange or open web steel joist chord supporting the *composite steel floor deck*.
2. The term “stud” shall refer to a steel headed stud anchor and shall be not be less than 3/8 in. (10 mm) nor greater than 7/8 in. (22 mm) in diameter.
3. 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.
4. Studs shall be installed as required by Section F3.2.3.3.
5. The *deck* section depth, d_d, shall not be less than 1.5 in. (37 mm) nor greater than 6.25 in. (160 mm).

6. The *rib pitch* shall not be less than 6 in. (152 mm) nor greater than 14 in. (356 mm).
7. The base metal thickness, t, shall not be less than 0.028 in. (0.71 mm) nor greater than 0.075 in. (1.91 mm).
8. The concrete thickness above the *deck*, h_c, shall not exceed 6 in. (152 mm) for the purposes of calculating yield moment, M_{nu}.

User Note: The limitation of 6 in. concrete thickness above the *steel deck*, h_c, is only a limitation of calculating M_{nu} and does not prohibit greater depths of concrete cover for purposes other than calculating M_{nu}.

The following safety factors, Ω (ASD) and resistance factors, ϕ (LRFD) shall be applied to the nominal moment capacity calculated for the *composite steel deck-slab*, for the applicable limit state(s):

Limit State	Ω (ASD)	ϕ (LRFD)
Flexure (under-reinforced)	1.75	0.85

F3.2.3.2 Flexural Strength

This section shall be used to determine the flexural strength of the *composite steel deck-slab*.

1. 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 steel deck-slab*

= F_T / Q_n (Eq. F3.2-14)

Where:

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

= $F_y (A_s - 0.5A_{webs} - A_{bf})$ (Eq. F3.2-15)

Where:

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

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

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

Q_n = nominal shear strength of steel headed stud anchor

= $0.5A_{sa} (E_c f'_c)^{0.5} \leq 0.75A_{sa}F_{us}$ (Eq. F3.2-16)

Where:

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

F_{us} = Specified minimum tensile strength of a steel headed stud anchor

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

$$M_n = M_{nu} = A_sF_y(d - a/2) \qquad \text{(Eq. F3.2-17)}$$

Where:

a = developed depth of concrete in the compression zone

= $A_sF_y / (0.85f'_c b)$ (Eq. F3.2-18)

b = unit width of compression face of *composite steel deck-slab*

d = distance from extreme compression fiber of *composite steel deck-slab* to centroid of the *steel deck*

f'_c = concrete strength, ksi

F_y = yield stress of *steel deck*

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

$$M_n = M_{no} + (M_{nu}-M_{no})(N_s/ N_{su}) \qquad \text{(Eq. F3.2-19)}$$

Where:

M_{nu} = as calculated by Eq. F3.2-17

M_{no} = nominal moment capacity of the *composite steel deck-slab*, calculated by either Section F3.2.1 or Section F3.2.2.

N_s = number of studs installed per unit width

F3.2.3.3 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.
4. The distance of the stud to the end of a *deck panel*, l_e, measured from the center of the stud, shall not be less than:

$$\ell_e \geq 1.5 \text{ (stud diameter + 1/8 in. (3mm))} \qquad \text{(Eq. F3.2-20)}$$

User Note: Figures illustrating proper stud installation are found in the Commentary.

F4 One-Way Shear Strength

The one-way shear strength of the *composite steel deck-slab* shall be calculated as follows:

LRFD:

$\phi\ V_n = \phi_c V_c + \phi_s V_D \leq 0.004\ \phi_c\ A_c(f'_c)^{0.5}$ (Eq. F4-1a) (USCS)

$\phi\ V_n = \phi_c V_c + \phi_s V_D \leq 0.332\ \phi_c\ A_c(f'_c)^{0.5}$ (Eq. F4-1b) (SI)

ASD:

$V_n/\Omega = V_c/\Omega_c + V_D/\Omega_s \leq 0.004 A_c(f'_c)^{0.5} / \Omega_c$ (Eq. F4-2a) (USCS)

$V_n/\Omega = V_c/\Omega_c + V_D/\Omega_s \leq 0.332 A_c(f'_c)^{0.5} / \Omega_c$ (Eq. F4-2b) (SI)

Where:

V_c = nominal one-way shear resistance of concrete

$V_c = 2\ \lambda(f'_c)^{0.5}\ A_c$ (Eq. F4-3a) (USCS)

$V_c = 0.166\ \lambda(f'_c)^{0.5}\ A_c$ (Eq. F4-3b) (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), calculated as the concrete contained within the webs of the *deck* and the projection of the webs into the slab above the *deck*. If the slab depth causes the projected area to overlap, the area is adjusted to not exceed the shape provided, with the *deck pitch* as the top dimension.

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).

The following safety factors, Ω (ASD) and resistance factors, ϕ (LRFD) shall be applied to the nominal shear capacity calculated for the *composite steel deck-slab*, for the applicable limit state(s):

Limit State	Ω (ASD)	ϕ (LRFD)
Resistance of Concrete (Ω_c , ϕ_c,)	2.00	0.75
Resistance of Steel Deck (Ω_s , ϕ_s,)	1.75	0.85

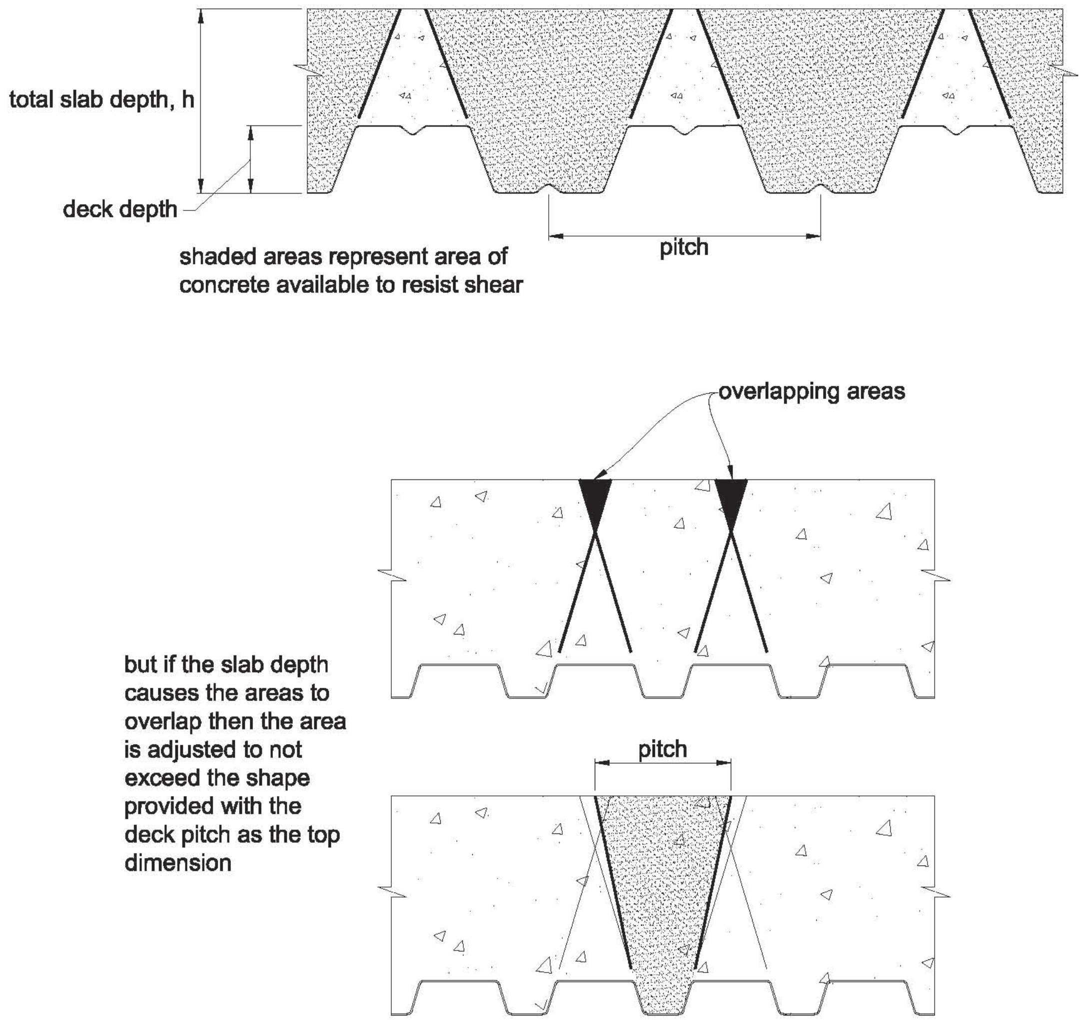


Figure F4-1 – One-Way Shear Parameters - Web Angle < 90 Degrees

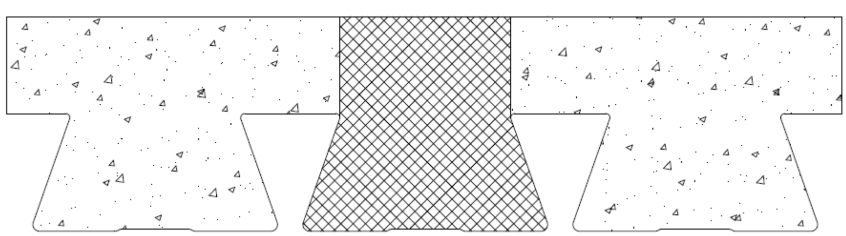


Figure F4-2 – One-Way Shear Parameters - Web Angle ≥ 90 Degrees

User Note: The area of the concrete, A_c, is illustrated in Figures F4-1 and F4-2. It 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 A_c in Equations F4-1, F4-2 and F4-3. The vertical shear strength of the *composite steel deck-slab* is determined for the *pitch* width 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*.

F5 Two-Way Shear Strength

The two-way shear resistance, V_{pr}, shall be determined as follows:

$$V_{pr} = 0.001(2+4/\beta_c)b_oh_c(f'_c)^{0.5} \le 0.004b_oh_c(f'_c)^{0.5} \quad \text{(Eq. F5-1a) (USCS)}$$

$$V_{pr} = 0.043(2+4/\beta_c)b_oh_c(f'_c)^{0.5} \le 0.332b_oh_c(f'_c)^{0.5} \quad \text{(Eq. F5-1b) (SI)}$$

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

The following safety factors, Ω (ASD) and resistance factors, ϕ (LRFD) shall be applied to the nominal shear resistance calculated for the *composite steel deck-slab*, for the applicable limit state(s):

Limit State	Ω (ASD)	ϕ (LRFD)
Two-Way Shear	2.00	0.75

The critical surface for calculating two-way shear shall be perpendicular to the plane of the slab and located outside of the periphery of the concentrated load or reaction area. 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.

F6 Lateral Distribution of Concentrated Loads

Concentrated loads on *composite steel deck-slabs* shall be permitted to be laterally distributed perpendicular to the *deck ribs* by methods of rational analysis.

User Note: One acceptable rational method is found in Commentary Section F6.

When two-way flexural action is required for concentrated load distribution, the flexural strength of the concrete in the weak axis direction shall be determined using ACI 318.

F7 Negative Moments over Supports

Unless specifically designed for negative moments and detailed as a series of continuous spans, the *composite steel deck-slab* shall be considered to be a series of simply supported spans.

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.

F8 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.

Exception: The *deck* shall be permitted to carry all required dead and live loads, without consideration of contribution of the concrete slab.

F9 Reinforcement for Temperature and Shrinkage

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. **Reinforcing Steel**

   Grade 60 or higher welded wire reinforcement or Grade 60 or higher 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 less than the area provided by 6 x 6 – W1.4 x W1.4 (152 x 152 – MW9 x MW9) welded wire reinforcement. When Grade 40 or Grade 50 reinforcement is used, the minimum area of reinforcement shall be increased by 11%. Reinforcing bars or wire reinforcement shall be spaced no further apart than 5 times the thickness of the concrete above the top of the *deck*, nor farther apart than 18 inches (460 mm).

2. **Discontinuous Fibers**

   Concrete specified in accordance with ASTM C1116, containing steel fibers meeting the criteria of ASTM A820, Type I , Type II, or Type V, or macrosynthetic fibers meeting the criteria of ASTM D7508 for macro-chopped strands or hybrid chopped strands, or a blend of steel and synthetic and macrosynthetic fibers meeting these Standards shall be permitted if acceptable to the *Designer* and *Authority Having Jurisdiction*. If used, fibers shall be included at a dosage rate not less than 25 lbs/cu yd (14.8 kg/cu meter) for steel fibers, and not less than 4 lbs/cu yd (2.4 kg/m3) for macrosynthetic fibers.

When the slab is designed for negative moments in accordance with Section F7, temperature and shrinkage reinforcement shall be provided in accordance with Section F9. Section 7.6.4 of ACI 318 shall not apply.

User Note: Extensive discussion of this topic is found in the Commentary.

F10 Deflection

F10.1 Moment of Inertia

Composite steel deck-slab moment of inertia shall be calculated using methods of rational analysis applicable to cracked concrete cross sections.

User Note: One method of rational analysis is shown in Commentary Section F10.

F10.2 Time Dependent Deflections

F10.2.1 Slabs Without Additional Reinforcing Steel in Compression Zone

Unless obtained from a more comprehensive analysis, additional time dependent deflection resulting from concrete creep and shrinkage shall be calculated as the product of the immediate deflection caused by sustained load and the following factors:

(1.0) for load duration of 3 months
(1.2) for load duration of 6 months
(1.4) for load duration of 1 year
(2.0) for load duration of 5 years or longer.

F10.2.2 Slabs With Additional Reinforcing Steel in Compression Zone

Unless obtained from a more comprehensive analysis, additional time dependent deflections resulting from concrete creep and shrinkage shall be permitted to be calculated in accordance with ACI 318, Section 24.2.4.1.1. Alternately, the effect of this compression steel may be ignored and additional deflections may be calculated in accordance with Section F10.2.1

F10.2.3 Reduction of Additional Deflections

When checking the effect of deflections on nonstructural elements, additional deflections calculated in accordance with Sections F10.2.1 or F10.2.2 shall be permitted to be reduced by the amount of deflection calculated to occur before attachment of nonstructural elements.

SECTION G - Diaphragms

G1 Diaphragm Design

Diaphragm strength and stiffness shall be determined in accordance with AISI S310, or other methods permitted by the applicable building code.

SECTION H - Design of Connections

H1 General

Deck and *accessories* shall be attached to supports to resist required loads and to provide structural stability for the supporting member (if required). Connections shall be designed in accordance with AISI S100 or other applicable Standards, 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.

H2 Welding

H2.1 Controlling Standard

All welding of *deck* shall be in accordance with AWS D1.3.

H2.2 Weld Washers

For connection of the *deck* to the supporting structure, weld washers shall be used with arc spot welds on all *deck* units with base steel thickness less than 0.028 inches (0.71 mm). Weld washers shall be a minimum thickness of 0.050 inches (1.27 mm), a maximum thickness of 0.080 inches (2.03 mm) and have minimum 3/8 inch (10 mm) diameter hole. Weld washers shall not be used between supports along the *side-laps*.

User Note: AWS D1.3 and AISI S100 do not require the use of weld washers for welding sheet steel 0.028 inches (0.71 mm) and thicker. *Deck* meeting SDI recommended 22 *gage* (0.0295 inch design thickness) or thicker therefore do not require weld washers.

H2.3 Welding without Weld Washers

Where weld washers are not required, a minimum 5/8 inch (16 mm) visible diameter arc spot weld shall be used for total sheet thickness less than 0.045 inches (18 *gage*) and a minimum 3/4 inch (19 mm) visible diameter arc spot weld shall be used for total sheet thickness equal to or greater than 0.045 inches. Alternately, other welds sized to provide equivalent shear and tension resistance per AISI S100, may be substituted. Weld metal shall penetrate all layers of *deck* material at end and *side-laps* and shall have fusion to the supporting members as required by AWS D1.3.

User Note: The effect of total sheet thickness at *deck* laps should be considered when selecting weld diameters. Refer to AISI S100 for converting visible to effective diameter.

H2.4 Fillet Welds

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

H2.5 Bearing Surfaces

Deck bearing surfaces to be welded shall be brought into contact as required by AWS D1.3, Section 7.3.2.

H3 Mechanical Fasteners

H3.1 Screws

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

H3.2 Power-Actuated Fasteners

When the support fasteners are power-actuated fasteners, 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*.

H3.3 Alternate Mechanical Fasteners

Other fasteners shall be permitted when acceptable to the *Designer* and *Authority Having Jurisdiction*. The use of alternate fasteners shall be based on the manufacturers' applicable fastener test report or other documentation acceptable to the *Designer* and *Authority Having Jurisdiction*.

H4 Steel Headed Stud Anchors

When steel headed stud anchors are installed through the *steel deck* to the supporting beam or joist, the steel headed stud anchor shall be permitted as a substitute for a fastener to the supporting structure, subject to minimum fastener edge distance requirements for steel headed stud anchors in accordance with Eq. F3.2-18. Steel headed stud anchors shall be installed in accordance with AWS D1.1 when welded to steel beams or steel edge supports, or SJI 200 when welded to composite open web steel joists.

SECTION I - Manufacturing Tolerance

I1 Tolerance of Delivered Material

I1.1 Base Metal Tolerance

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 thickness 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.

I1.2 Embossment Tolerance

The average depth of *embossments* in *composite steel floor deck* (if any) shall not be less than 90% of the design *embossment* depth.

SECTION J - Installation

J1 Deck Fastening and Attachment

J1.1 Deck Support Attachment

Steel deck shall be anchored to structural supports using welds, steel headed stud anchors, or mechanical fasteners. Attachment to supports shall be capable of resisting the applied loads. Spacing of *deck* attachments at supports perpendicular to the span of the *deck panel* shall not exceed 18 inches (460 mm), unless otherwise permitted by the *Designer* and *Authority Having Jurisdiction*.

EXCEPTION: For *deck* resisting component and cladding uplift wind pressure in hurricane prone regions as defined in ASCE 7, the spacing of *deck* attachments at supports shall not exceed 12 inches (306 mm), except when *deck rib pitch* exceeds 12 inches (306 mm), where the spacing shall not exceed the *rib pitch*.

The *deck* shall be adequately attached to the structure to prevent the *deck* from slipping off the supporting structure.

User Note: ASCE 7 defines hurricane prone regions as areas where the basic wind speed for Risk Category II buildings is greater than 115 mph (51 m/s), located on the U.S. Atlantic Ocean and Gulf of Mexico coasts, Hawaii, Puerto Rico, Guam, Virgin Islands, and American Samoa.

J1.2 Deck Side-Lap Fastening

Fastening at the *deck side-lap* shall be designed and specified as follows:

For *deck* with spans less than or equal to 5 feet (1.5 m), *side-lap fasteners* shall not be required, unless required for diaphragm design.

For *deck* with spans greater than 5 feet (1.5 m), *side-laps* shall be fastened at intervals not to exceed 36 inches (1 m) on center, unless more frequent fastener spacing is required for diaphragm design.

Side-lap fastening shall be by one of the following methods:

1. Screws with a minimum diameter of 0.190 inches (4.83 mm) (#10 diameter)
2. Punch system or *button punch*
3. Arc spot welds in accordance with Section H2.3 or fillet welds in accordance with Section H2.4, or other weld shown to be substantially equivalent through testing in accordance with AISI S905, or by calculation in accordance with AISI S100.
4. Other *side-lap* attachment methods acceptable to the *Designer* and *Authority Having Jurisdiction*.

J1.3 Deck Perimeter Attachment Along Edges Between Supports

Attachment at the perimeter of the *deck* parallel to the *deck ribs* shall be designed and specified as follows:

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.

Perimeter attachment shall be by one of the following methods:

1. Screws with a minimum diameter of 0.190 inches (4.83 mm) (#10 diameter)
2. Arc spot welds in accordance with Section H2.2 or H2.3 or fillet welds in accordance with Section H2.4.
3. Power-actuated fasteners.
4. Steel headed stud anchors in accordance with Section H2.5.
5. Other methods acceptable to the *Designer* and *Authority Having Jurisdiction*.

J1.4 Edge Distance

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

J1.5 Bearing Contact

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

J2 Accessory Attachment

Structural *accessories* shall be attached to supporting structure or *deck* as required for transfer of forces, but not to exceed 18 inches (460 mm) on center, unless otherwise permitted by the *Designer* and *Authority Having Jurisdiction*. Non-structural *accessories* shall be attached to supporting structure or *deck* as required for serviceability, but not to exceed 24 inches (600 mm) on center unless otherwise permitted by the *Designer* and *Authority Having Jurisdiction*. Mechanical fasteners or welds shall be permitted for *accessory* attachment.

J3 Cleaning Prior to Concrete Placement

Surfaces where concrete will be placed shall be cleaned of debris prior to concrete placement.

J4 Reinforcing Steel and Imbedded Items

J4.1 Imbedded Materials

Reinforcing steel and any items to be imbedded into concrete shall be installed as required by the construction documents.

J4.2 Conduits

Conduits are permitted in non-composite concrete slabs as permitted by ACI 318, Section 20.7.

Unless otherwise shown to be acceptable by rational engineering analysis, conduits are only permitted in *composite steel deck-slabs* with the following restrictions:

1. The conduit must be steel or iron without an aluminized coating.
2. The maximum conduit diameter shall not exceed the lesser of 1 inch (25 mm) or 1/3 of the thickness of the concrete over the top of the *composite steel floor deck*.
3. The conduit shall have a minimum concrete cover of 3/4 inch (20 mm) between the top of the conduit and the top of the slab.
4. No crossovers of conduit shall be permitted.

J4.3 Aluminum Items

Items with aluminum surfaces shall not be imbedded into concrete unless properly isolated to prevent galvanic action.

J5 Temporary Shoring

Temporary shoring, if required for *deck* carrying fluid concrete loads, shall be designed to resist the loads indicated in Section C1. 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 a minimum of 75% of its specified design strength.

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

SECTION K - Steel Deck and Steel Deck-Slabs in Existing Structures

K1 Existing Structures

Steel deck and *composite steel deck-slabs* in existing structures shall be permitted to be analyzed either by this Standard, or by a previously existing Standard in effect at the time that the structure was designed, or other method acceptable to the *Authority Having Jurisdiction*.

APPENDIX 1 - Stainless Steel Deck

1.1 Stainless Steels for Applications other than Composite Steel Deck-Slabs

Stainless steels are permitted to be used for *steel deck* in the following applications:

a. *Steel deck* without cementitious topping
b. *Steel deck* with cementitious topping without composite *deck* action

The following substitutions within the Standard shall be made:

a. Section A3.1.1 is replaced in its entirety as follows

A3.1.1 Sheet Steel for Deck and Accessories that Carry Defined Loads

Sheet steel for *deck* and *accessories* that carry defined loads shall conform to ASCE 8, Section 1.3.

a. All sheet steel used for this purpose shall have a minimum specified yield stress that meets or exceeds 33 ksi (230 MPa).
b. 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.

b. Section A3.1.3 is deleted

c. Section A3.2.3 is deleted

d. Section A4.2: Substitute ASCE 8 for AISI S100

e. Replace Section G1 as follows:

Diaphragm strength and stiffness shall be permitted to be determined in accordance with AISI S310, suitably modified for the properties of stainless steel and its fasteners, or other methods permitted by the *Authority Having Jurisdiction*.

f. Modify Section H as follows:

Section H1: Substitute ASCE 8 for AISI S100
Section H2: Delete entire section
Section H3: Add user note at end of section:

User Note: Specified fastener material should have equivalent corrosion resistance to the most corrosion-resistant metal being fastened in order to reduce the likelihood of galvanic corrosion that may occur when dissimilar metals are connected.

Section H4: Delete entire section

g. Modify Section I as follows:

Section I1.1: Substitute ASCE 8 for AISI S100
Section I1.2: Delete entire section

h. Modify Section J as follows:

Delete all references to welds.
Section J1.4: Substitute ASCE 8 for AISI S100

1.2 Stainless Steels for Composite Steel Deck-Slabs

Stainless steels shall be permitted to be used for *composite steel deck-slabs* only under one of the following conditions:

1. Full scale performance testing as per SDI T-CD is performed; or
2. Other testing or analysis methods acceptable to the authority having jurisdiction are performed.

APPENDIX 2 - Construction Phase Loads - Strength and Deflection

2.1 Deck Supporting Fluid Concrete

2.1.1 Supported Spans

Deck shall be evaluated for strength under the following loads and load combinations:

a. Allowable Strength Design

$$w_{dc} + w_{dd} + w_{lc} \qquad \text{(Eq. 2.1-1)}$$

$$w_{dc} + w_{dd} + P_{lc} \qquad \text{(Eq. 2.1-2)}$$

$$w_{dd} + w_{cdl} \qquad \text{(Eq. 2.1-3)}$$

b. Load and Resistance Factor Design

$$1.6w_{dc} + 1.2w_{dd} + 1.4w_{lc} \qquad \text{(Eq. 2.1-4)}$$

$$1.6w_{dc} + 1.2w_{dd} + 1.4P_{lc} \qquad \text{(Eq. 2.1-5)}$$

$$1.2w_{dd} + 1.4w_{cdl} \qquad \text{(Eq. 2.1-6)}$$

Where:

w_{dc} = dead weight of fluid concrete

w_{dd} = dead weight of the *steel deck*

w_{lc} = uniform construction live load (combined with fluid concrete), minimum 20 psf (0.96 kPa)

w_{cdl} = uniform construction live load (combined with bare *deck*), minimum 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: Refer to the Commentary for extensive discussion of construction live loads and the types of equipment used for concrete placement and finishing.

2.1.2 Cantilever Spans

Cantilever spans shall be evaluated for strength under the following load combinations:

1. Allowable Strength Design: Equations 2.1-1 and 2.1-2 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.
2. Load and Resistance Factor Design: Equations 2.1-4 and 2.1-5 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.

2.1.3 Loading Considerations

1. The specified construction live loads shall be increased when required by the construction operations.
2. Loads shall be applied in a sequence that simulates the placement of the concrete. Loads shall be applied on spans in such a way that the maximum moment, shear, deflections, and support reactions are determined. Rational analysis shall be permitted to be used for developing shear and moment diagrams and calculating deflections for uniform and non-uniform spans.

User Note: Figures in Commentary for Appendix Section 2.1.3 show the rational application of loads to develop the maximum moment, shear, deflections, and support reactions.

2.1.4 Deck Deflection

1. Calculated deflections of the *deck* as a form shall be based on the load of the fluid concrete as determined by the design slab thickness and the self-weight of the *steel deck*, uniformly loaded on all spans. First order elastic deflection shall be limited to 1/180 of the clear span. Calculated deflections shall be relative to supporting members.
2. If the maximum first order *deck* deflection under the dead load of the design slab thickness and self-weight of the *steel deck* exceeds the greater of 1/10 of the total design slab depth or 3/4 inch (19 mm), the additional concrete weight due to the *deck* deflection shall be taken into consideration in the design of the *deck* and its supporting members. The effect of the additional concrete shall be permitted to be considered by increasing the nominal slab depth by 0.7 times Δ, over whole span, where Δ is the maximum first order elastic *deck* deflection. Other methods of rational analysis shall be permitted which take into account the second-order effects of concrete ponding.
3. The deflection of cantilevered *deck* as a form, as determined by slab thickness and self-weight of the *steel deck*, shall not exceed $L_c/90$, where "L_c" is the cantilever length, nor 3/4 inches (19 mm).

User Note: If the framing supporting the *steel deck* also deflects appreciably, then the effect of this deflection should be included in the calculations. Framing deflection can be adequately controlled by proper specification of camber of the framing members.

2.2 Deck Not Supporting Fluid Concrete

2.2.1 Supported Spans

Deck shall be evaluated for strength under the following loads and load combinations:

1. Allowable Strength Design

$w_{dd} + P_{lc}$ (Eq. 2.2-1)

$w_{dd} + w_{cdl}$ (Eq. 2.2-2)

2. Load and Resistance Factor Design

$1.2w_{dd} + 1.4P_{lc}$ (Eq. 2.2-3)

$1.2w_{dd} + 1.4w_{cdl}$ (Eq. 2.2-4)

Where:
w_{dd} = dead weight of the *steel deck*
P_{lc} = concentrated construction live load per unit width of *deck* section; 200 pounds on a 1 foot width (2.92 kN on a 1 meter width)
w_{cdl} = uniform construction live load (combined with bare *deck*), minimum 20 psf (0.96 kPa)

User Note: The minimum 20 psf construction live load is for *deck* used as a permanent floor, roof or similar surface in a building structure. The minimum loads for *steel deck* used as a component in non-permanent construction platforms, scaffolding or other temporary structures should be designed to resist the minimum loads required by applicable safety standards.

2.2.2 Cantilever Spans

Cantilever spans shall be evaluated for strength under the following load combinations:

1. Allowable Strength Design

$w_{dd} + w_{lcc} + P_{lc}$ (Eq. 2.2-5)

2. Load and Resistance Factor Design

$1.2w_{dd} + 1.6w_{lcc} + 1.4\ P_{lc}$ (Eq. 2.2-6)

Where:

w_{dd} = dead weight of the *steel deck*

w_{lcc} = uniform construction live load applied to cantilever span and adjacent span, 10 psf (0.48 kPa)

P_{lc} = concentrated construction live load per unit width of *deck* section; 200 pounds on a 1 foot width (2.92 kN on a 1 meter width), applied at the end of the cantilever

2.2.3 Loading Considerations

The specified construction live loads shall be increased when required by the construction operations.

2.2.4 Deck Deflection

Load combinations in Sections 2.2.1 and 2.2.2 shall not be subject to deflection limitations.

User Note: The load combinations of these Sections are temporary construction loads and should not be limited to deflections intended to provide serviceability in the completed structure.

COMMENTARY

(The Commentary is not a part of SDI SD-2022 Standard for Steel Deck, but is included for informational purposes only.)

Introduction

The Standard is intended to be complete for normal usage.

The Commentary furnishes background information and references for the benefit of the user seeking further understanding of the basis, derivations and limits of the Standard.

The Standard and Commentary are intended for use by users with demonstrated engineering competence.

Commentary Symbols

The Commentary uses the following symbols in addition to the symbols defined in the Standard. The section number in the right-hand column refers to the Commentary section where the symbol is first used.

Symbol	Definition	Section
A_s	*Steel deck* cross sectional area per unit width of *steel deck*	F2.2
E_c	Modulus of elasticity of concrete	F2.3
E_s	Modulus of elasticity of *steel deck*	F2.2
I_{cr}	Cracked moment of inertia transformed to steel	F2.3
I_D	Moment of inertia for deflection calculation under uniform load	A4.2
I_n	Effective moment of inertia, negative bending	A4.2
I_p	Effective moment of inertia, positive bending	A4.2
I_{sf}	Moment of inertia of the full (unreduced) *steel deck* per unit slab width.	F2.3
I_u	Uncracked moment of inertia transformed to steel	F2.3
I_x	Full (gross) moment of inertia	A4.2
b	Unit width of compression face of composite slab, 12 in.(1000 mm)	F2.2
c	Distance from extreme compression fiber to composite neutral axis, in. (mm)	F2.2
h	Nominal out-to-out depth of slab, in. (mm)	F2.2
n	Modular ratio	F2.3
w_c	Concrete unit weight, pcf (kg/m^3)	F2.3
ρ	Reinforcing ratio	F2.3

SECTION A - General Provisions

A1 Scope, Applicability, and Definitions

The scope of this Standard is essentially the same as the combined scopes of the 2017 Standard for Steel Roof Deck (SDI RD-2017), the 2017 Standard for Non-Composite Floor Deck (SDI NC-2017), and the 2017 Standard for Composite Steel Floor Deck-Slabs (SDI C-2017), that this Standard replaces.

This Standard is not intended to apply to metal panel roof or wall systems constructed of cold-formed steel, where the roof or wall panel acts as the roof or wall covering providing both weather protection and support for structural loads. These panels are commonly used as roof or wall panels over open framing in pre-engineered buildings, post-frame buildings, and canopy structures.

A2 Reference Codes, Standards, and Documents

The following documents are referenced within the Commentary:

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-15, Guide for Concrete Floor and Slab Construction
   c. ACI 224.1R-07, Causes, Evaluation, and Repair of Cracks in Concrete Structures
   d. ACI 318-19, 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 Iron and Steel Institute (AISI)
   a AISI S100-16 w/S2-20 (2020), North American Specification for the Design of Cold-Formed Steel Structural Members
   b. AISI S310-20, North American Standard for the Design of Profiled Steel Diaphragm Panels
   c. AISI S907-17, Test Standard for Cantilever Test Method for Cold-Formed Steel Diaphragms
4. American Institute of Steel Construction (AISC)
   a. AISC Design Guide No. 11, Floor Vibrations Due to Human Activity, 2003
   b. AISC Design Guide No. 18, Steel-Framed Open-Deck Parking Structures, 2003
   c. ANSI/AISC 360-22, Specification for Structural Steel Buildings

5. American Society for Testing and Materials (ASTM)
   a. ASTM A653 / A653M - 17 Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process
   b. ASTM A1008 / A1008M - 16, 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 C1609 / C1609M - 12 Standard Test Method for Flexural Performance of Fiber-Reinforced Concrete (Using Beam With Third-Point Loading)
   d. ASTM E119 - 18, Standard Test Methods for Fire Tests of Building Construction and Materials
6. American Society of Civil Engineers (ASCE)
   a. ANSI/ASCE 3-91, Standard for the Structural Design of Composite Slabs.
   b. ASCE/SEI 8-21, Specification for the Design of Cold-Formed Stainless Steel Structural Members
   c. ASCE 9-91, Standard Practice for the Construction and Inspection of Composite Slabs
7. European Committee for Standardization (CEN)
   a. EN 1994-1-1:2004, Eurocode 4: Design of Composite Steel and Concrete Structures – Part 1-1: General rules and rules for buildings
8. Standards Australia/Standards New Zealand
   AS/NZS 2327:2017, Australian/New Zealand Standard, Composite Structures—Composite Steel-Concrete Construction in Buildings
9. Steel Deck Institute (SDI)
   a. SDI Internal Report: Luttrell, L.A. (2000), Composite Deck Formulations
   b. SDI C-2006, Standard for Composite Steel Floor Deck-Slabs
   c. SDI C-2011, Standard for Composite Steel Floor Deck-Slabs
   d. SDI C-2017, Standard for Composite Steel Floor Deck-Slabs
   e. SDI COSP-2017, Code of Standard Practice
   f. SDI Composite Deck Design Handbook, 2nd Edition (1997)
   g. SDI-DDM04, Diaphragm Design Manual, 4th Edition
   h. SDI-FDDM02, Floor Deck Design Manual, 2nd Edition
   i. SDI-FCASD, Fundamentals of Corrosion and Their Application to Steel Deck
   j. SDI-MOC, Manual of Construction with Steel Deck, 3rd Edition
   k. SDI NC-2017, Standard for Non-Composite Floor Deck
   l. SDI Position Statement "Use of Composite Steel Floor Deck in Parking Garages"
   m. SDI RD-2017, Standard for Steel Roof Deck
   n. SDI-RDDM02, Roof Deck Design Manual, 2nd Edition
   o. SDI- SDCFSFDM, Steel Deck on Cold-Formed Steel Framing Design Manual
10. UL, LLC (UL)
   a. Fire Resistance Directory

11. Other Publications
    a. Tremblay, R., Rogers, C.A., Gignac, P. and Degrange, G., (2002) "Variables Affecting the Shear-bond Resistance of Composite Floor Deck Systems," Proceedings of the 16th International Specialty Conference on Cold Formed Steel Structures.
    b. Widjaja, B.R. and Easterling, W.S. (1996), "Strength and Stiffness Calculation Procedures for Composite Slabs," Proceedings of 13th International Specialty Conference on Cold Formed Steel Structures.

A3 Material

A3.1 Applicable Steels

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 B1 of this Standard).

For *floor decks*, although *deck* painted on the top may be used as a form, uncoated steel or a galvanized surface is typically used in contact with concrete to achieve better chemical bond.

A3.2 Concrete and Reinforcement

The minimum concrete compressive strength for structural concrete slabs designed in accordance with ACI 318 is 2500 psi. The applicable building code may have other more stringent requirements on concrete strength.

Most load tables for *composite steel deck-slabs* are based on a 3000 psi concrete strength; however, some tables may require a higher concrete strength. Per the Standard, the maximum concrete strength used for calculation of the *composite steel deck-slab* is 6000 psi, even if the concrete strength is greater than 6000 psi, unless justified by testing. Increases in the concrete strength have little effect on the nominal strength of the *composite steel deck-slab*, however, the higher modulus of elasticity of higher strength concretes does have a beneficial effect in reducing deflections.

The calculated flexural strength of *composite steel deck-slabs* is little influenced by concrete strength (f'_c) and testing and research has shown that the concrete-*deck* bond is likewise unaffected by concrete strength. If the concrete compressive strength of a *composite steel deck-slab* determined by cylinder testing falls lower than specified, the strength of the *composite steel deck-slab* can be recalculated at the lower strength, which will usually prove to be adequate. Section 26.12.3.1 of ACI 318 considers concrete to be acceptable if the following is attained:

(1) The arithmetic average of any 3 consecutive strength tests equals or exceeds f'_c, and
(2) No strength test falls below f'_c by more than 500 psi if f'_c is 5000 psi or less, or by more than 0.10 f'_c if f'_c exceeds 5000 psi.

Labeled fire resistant rated assemblies may require concrete compressive strengths in excess of 3000 psi. Many assemblies require 3500 to 4000 psi or higher concretes, along with tolerances on the concrete unit weight. The *Designer* should be aware of these requirements and specify the concrete accordingly.

A4 Design Basis (Required Strength, ASD, LRFD)

A4.1 Required Strength

This Standard permits two methods of design:

(a) Load and Resistance Factor Design (LRFD): The nominal strength is multiplied by a resistance factor, ϕ, resulting in the design strength, which is then required to equal or exceed the required strength determined by structural analysis for the appropriate LRFD load combinations specified by the applicable building code.

(b) Allowable Strength Design (ASD): The nominal strength is divided by a safety factor, Ω, resulting in the allowable strength, which is then required to equal or exceed the required strength determined by structural analysis for the appropriate ASD load combinations specified by the applicable building code.

This Standard gives provisions for determining the values of the nominal strengths according to the applicable limit states and lists the corresponding values of the resistance factor, ϕ, and the safety factor, Ω. Nominal strength is usually defined in terms of resistance to a load effect, such as axial force, bending moment, shear or torque, but in some instances it is expressed in terms of a stress. The term available strength is used throughout the Standard to denote design strength and allowable strength, as applicable.

A4.2 Section Properties and Deck Strength

A generally accepted method of rational analysis for perforated *deck* is found in the SDI publication "Perforated Metal Deck Design with Commentary." Perforations for venting or hanger tabs, which generally result in a less than 2% open area, can be neglected in design.

The estimated deflection of *steel deck* is in relationship to the effective section properties and the applied bending stress on the member. In regions of the *deck* where the bending stress is low, the effective section properties will often be equal to the full (gross) section properties. In regions of the *deck* where the applied bending stress approaches the service load bending stress, the effective properties will normally be lower than the full section properties. Along the span of the *deck*, the applied bending stress varies, therefore the effective section properties vary along the span of the *deck*. This would be analogous to continually changing section properties of a tapered beam.

For uniformly distributed loads on *steel deck*, the following weighted averages of the full and effective section properties provide a good estimate of an average equivalent effective section property for the estimation of deflection under service loads.

Simple span: $I_D = (I_x + 2I_p) / 3$

Multiple span: $I_D = (I_x + 2I_n) / 3$ if negative bending controls, or
$I_D = (I_x + 2I_p) / 3$ if positive bending controls.

Where:
I_D = Moment of inertia for deflection calculation under uniform load
I_x = Full (gross) moment of inertia
I_p = Effective moment of inertia, positive bending
I_n = Effective moment of inertia, negative bending

A5 Specification of Sheet Steel Thickness

Both AISI and SDI now specify steel thickness in terms of design thickness in lieu of *gage* thickness. *Gage* thickness, however, is still commonly referred to in the *steel deck* industry. Table C-A5-1 shows common *gages* and corresponding uncoated design and minimum steel thickness. A more extensive table is contained in the SDI-COSP.

Table C-A5-1

Gage No.	Design Thickness		Minimum Thickness[1]	
	in.	mm.	in.	mm.
26	0.0179	0.45	0.0170	0.43
24	0.0238	0.60	0.0226	0.57
22	0.0295	0.75	0.0280	0.71
20	0.0358	0.91	0.0340	0.86
18	0.0474	1.20	0.0450	1.14
16	0.0598	1.52	0.0568	1.44

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

A6 Definitions

Terms which are not defined in the Standard or AISI S100 have the ordinary accepted meaning for the context for which they are intended. The Commentary uses the following terms in addition to the terms defined in the Standard.

Center-to-Center Span**:** The distance between the centerline of supporting structural members.

Clear Span**:** The actual clear distance or opening between supporting structural members, i.e., the distance between wall faces or the distance between the edges of adjacent beam flanges.

Owner's Designated Representative for Construction: The owner or the entity that is responsible to the owner for the overall construction of the project, including its planning, quality, and completion.

SECTION B - Construction Documents

B1 Construction Documents

The following are examples of how various types of *deck* and *deck* applications could be specified within the contract documents:

Roof Deck

1. Loads
   a. Roof Dead Load = 15 psf (ASD)
      Roof Live Load = 20 psf (ASD)
      Snow Load = 15 psf (ASD)
      Roof Wind Load = See wind pressure diagram
   b. Assumed construction phase load = 20 psf (ASD)
   c. Diaphragm Loading = See diaphragm loading diagram
2. *Roof Deck* and *Deck* Attachment
   a. 1.5 WR, 22 *gage* (0.0295 inch base steel design thickness)
   b. Minimum F_y = 33 ksi, Minimum F_u = 45 ksi, coated with manufacturer's standard shop coat
   c. Attach *deck* to support framing using a 36/5 pattern with #12 screws. Attach *side-laps* at 16 inches on center using #10 screws. Attach *deck ribs* parallel to support at a collector using #12 screws at 12 inches on center.

Composite Steel Floor Deck

1. Loads
   a. Floor Dead Load = 63 psf (ASD)
      Dead weight of concrete = 60 psf
      Dead weight of the *steel deck* = 3 psf
      Floor Live Load = 100 psf (ASD)
   b. Assumed construction phase loads. The following construction phase loads were used by the *Designer* in selecting the *deck* profile. It is the responsibility of the *Owner's Designated Representative for Construction* to determine the adequacy of these loads for the intended construction operations, and to submit, in writing, confirmation of this to the *Designer* prior to beginning construction operations.
      w_{lc} (uniform construction live load, combined with fluid concrete) = 20 psf
      w_{cdl} (uniform construction live load, combined with bare *deck*) = 50 psf
      P_{lc} (concentrated construction live load per unit width of *deck* section) = 150 pounds on a 1 foot width
   c. Diaphragm Loading = See diaphragm loading diagram
2. *Composite Steel Floor Deck* and *Deck* Attachment
   a. 2x12 composite *deck*, 18 *gage* (0.0474 inch base steel design thickness)
   b. Minimum F_y = 50 ksi, Minimum F_u = 60 ksi, G90 galvanized

c. Attach *deck* to support framing using a 36/4 pattern with 3/4 inch visible diameter arc spot welds (E60xx electrode). Attach *side-laps* at 24 inches on center using #10 screws. Attach *deck ribs* parallel to support at a collector using 3/4 inch visible diameter arc spot welds (E60xx electrode) at 12 inches on center.

3. Concrete and Reinforcing
    a. Specified concrete strength, f'_c = 3000 psi minimum
    b. Specified concrete density = normal weight concrete, 145 pcf nominal
    c. See drawings for all required reinforcing steel
    d. Slab thickness as indicated on drawings
    e. Concrete is to be placed to a uniform thickness
4. The requirement to determine temporary shoring (if required) is the responsibility of the *Owner's Designated Representative for Construction*
5. See drawings for required fire rated assembly

Non-Composite Steel Floor Deck

1. Loads
    a. Floor Dead Load = 53 psf (ASD)
        Dead weight of concrete = 50 psf
        Dead weight of the *steel deck* = 3 psf
       Floor Live Load = 100 psf (ASD)
    b. Assumed construction phase loads. The following construction phase loads were used by the *Designer* in selecting the *deck* profile. It is the responsibility of the *Owner's Designated Representative for Construction* to determine the adequacy of these loads for the intended construction operations, and to submit, in writing, confirmation of this to the *Designer* prior to beginning construction operations.
        w_{lc} (uniform construction live load, combined with fluid concrete) = 20 psf
        w_{cdl} (uniform construction live load, combined with bare *deck*) = 50 psf
        P_{lc} (concentrated construction live load per unit width of *deck* section) = 150 pounds on a 1 foot width
    c. Diaphragm Loading = See diaphragm loading diagram
2. *Non-Composite Steel Floor Deck* and *Deck* Attachment
    a. 1-1/2 inch x 6 inch *form deck*, 18 *gage* (0.0474 inch base steel design thickness)
    b. Minimum F_y = 40 ksi, Minimum F_u = 50 ksi, G60 galvanized.
    c. Attach *deck* to support framing using a 36/4 pattern with 3/4 inch visible diameter arc spot welds (E60xx electrode). Attach *side-laps* at 24 inches on center using #10 screws. Attach *deck ribs* parallel to support at a collector using 3/4 inch visible diameter arc spot welds (E60xx electrode) at 12 inches on center.
3. Concrete and Reinforcing
    a. Specified concrete strength, f'_c = 3000 psi minimum
    b. Specified concrete density = normal weight concrete, 145 pcf nominal
    c. See drawings for all required reinforcing steel
    d. Slab thickness as indicated on drawings
    e. Provide W2.9 x W2.9 - 6x6 WWR (mats only) for temperature and shrinkage reinforcement. Locate WWR 1 inch from top of slab.
    f. Concrete is to be placed to a uniform thickness

4. The requirement to determine temporary shoring (if required) is the responsibility of the *Owner's Designated Representative for Construction*
5. See drawings for required fire rated assembly

The equivalent information can be provided on either the project drawings or project *specifications*; however, care should be taken to coordinate the information. The preceding suggested *specifications* should be modified and added to as required to provide a complete specification of what is intended for the finished, installed product.

Deck Finish

The *finish* on the *steel deck* must be specified by the *Designer* and be suitable for the environment to which the *deck* is exposed within the finished structure. The uncoated *finish* is by custom referred to as "black" by some users and manufacturers; the use of the word "black" does not refer to paint color on the product.

When a primer coat is specified, 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. Field painting of primer coated *deck* is recommended especially where the *deck* is exposed. Light rusting on *panels* exposed to moisture should be expected.

In corrosive or high moisture atmospheres, a galvanized *finish* is desirable using a G60 (Z180) or G90 (Z275) 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. For *floor deck* applications in highly corrosive environments, consideration should be given to designing the concrete slab to resist all service loads.

Zinc-Aluminum *finishes* are not recommended for *deck* in contact with concrete. Serious consequences can arise in situations of long-term contact. A significant corrosion of aluminum embedded in concrete can occur. The corrosion can cause expansion of the concrete and subsequent cracking and de-bonding of hardened concrete. If the aluminum is coupled with any ferrous metals (the *steel deck*), galvanic corrosion will occur also. In both cases the presence of calcium chloride greatly accelerates the corrosion process.

If specifying painted *deck* in areas (both roof and floor) 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 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.

For *floor deck* applications, some *deck* manufacturing companies have concrete and steel floor units that are classified in some of the D700, D800, and D900-series designs in the UL Fire Resistance Directory. These classified *deck* units have been evaluated for use in these specific designs and found acceptable. These classified *deck* units (Classified Steel Floor and Form Units) are shown as having a galvanized *finish* or a phosphatized (or bonderized) surface preparation with an applied painted *finish*. UL listed assemblies often use the archaic term "phos/painted" or "phosphatized/painted" for a *finish* that has been prepared for painting using a phosphate to improve paint adherence, then painted. The Association for Iron and Steel Technology (AIST) currently used the term "bonderizing" as the treatment of cold rolled or galvanized steel surfaces with phosphate to improve paint adherence.

The following *finishes* are generally acceptable for normal environmental conditions:

Roof Deck:

Galvanized
Primer coated
Bare steel

Non-Composite Steel Floor (Form) Deck:

Galvanized
Primer coated bottom, bare top
Bare steel

Composite Steel Floor Deck:

Galvanized
Primer coated bottom, bare top
Bare steel
Primer coated or painted top surfaces are not recommended for *composite steel floor deck* due to the uncertainty of the resulting concrete to *deck* bond.

SDI-FCASD is a good reference regarding corrosion and the selection of *deck finish*.

SECTION C - Design of Steel Deck for Construction Phase Loads

Neither the *Designer* nor the *deck* manufacturer or *deck* supplier has any control over means and methods of construction, including the methods of concrete placement and finishing. For this reason, Section B1 requires that the *Designer* include the assumed construction loads in the construction documents and it is suggested that the construction documents require that the *Owner's Designated Representative for Construction* confirm in writing that the design live loads are adequate for all construction operations, including the concrete placement and finishing methods to be used, or that the *Owner's Designated Representative for Construction* must either shore the *deck* or increase the *deck* section (depth or *gage*) to accommodate the heavier construction loads being used.

SECTION D - Design of Steel Deck for In-Service Conditions - General Provisions

Traditionally the *clear span* is used for *floor deck* and *composite steel deck-slab* design, although using the span measured from center-to-center of supports is a conservative assumption.

For *roof deck*, one possible assumption is to consider the *clear span* for gravity load design, and the span measured from center-to-center of supports for uplift load design.

D1 Strength Design

In addition to uniformly distributed loads, loads may include concentrated loads, loads suspended from the *deck*, moving loads, and cyclic loads. These loads should be considered as applicable, because the controlling load case may include these loads.

The *Designer* should take into account the sequence of loading since the order in which spans are loaded, and the order in which loads are applied, may control the design moments, shears, or reactions.

Suspended loads may include ceilings, light fixtures, ducts or other utilities. The *Designer* should consider that loads could be applied directly to the *deck*, rather than the support framing.

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 hanger tabs or other hanging devices. Improper use of hanger tabs or other hanging devices could result in the overstressing of tab or device and/or the overloading of the *deck* or *composite steel deck-slab*.

D2 Fire Resistance

Fire rating requirements may dictate the concrete strength or density. Many fire rated assemblies that use *non-composite* or *composite steel floor decks* are available. In the Underwriters Laboratories *Fire Resistance Directory*, the *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.

D3 Deck Deflection

In absence of a building code, *roof deck* service load deflections should be limited to the deflections in Table C-D3-1.

Table C-D3-1 Maximum Service Load Deflection for Roof Deck [2]

Roof Deck Construction	Live Load	Snow or Wind[1]	Dead + Live Load
Supporting plaster ceiling	L/360	L/360	L/240
Supporting non-plaster ceiling	L/240	L/240	L/180
Not supporting ceiling	L/180	L/180	L/120

1. Ultimate wind loads shall be permitted to be multiplied by 0.42
2. For cantilever members, L shall be taken as twice the length of the cantilever

For purposes of Table C-D3-1, wind pressures calculated using ASCE 7-16 (and ASCE 7-10) are "ultimate" pressures. Wind pressures calculated using ASCE 7-05 or earlier editions are "nominal" and should be multiplied by 0.7 instead of 0.42.

The effect of *deck* deflections on insulation, roofing materials, and non-structural toppings, such as gypsum leveling concrete or lightweight insulating concretes, should be considered by the *Designer* to ensure compatibility of deflections with the materials proposed to be installed.

The adequacy of *roof deck* edge support details should be reviewed by the *Designer*. At the building perimeter or any other *deck* termination or direction change, occasional concentrated loading of the *roof deck* could result in temporary differences in deflection between the *roof deck* and the adjacent stationary building component. Supplemental support such as a perimeter angle may be warranted.

SECTION E - Design of Steel Deck with Cementitious Topping without Composite Deck Action for In-Service Conditions

E1 Structural Concrete Slab Design

E1.3 Concrete Thickness

The minimum concrete coverage of 1-1/2 inches is for structural design only. *Decks* that have wide top flanges may require thicker concrete cover to reduce the potential for cracking over the *deck* flange. Thicker concrete slabs may be required for fire rated floor assemblies.

E1.4 Calculation of Slab Superimposed Load Capacity

In normal conditions where the *non-composite steel floor deck* can reasonably be considered to not deteriorate over time, such as a floor in an enclosed building, the *deck* can be considered to be a permanent component, and the dead weight of the concrete may be considered to be carried by the *deck*. In an aggressive environment, such as in an open parking garage, where water and corrosive salts can reach the *steel deck*, or in areas of buildings where excessive moisture or corrosive fumes, such as chlorine or acids can attack the *deck*, it should be considered to be impermanent, and the dead weight of the concrete should be considered to be carried by the slab. In some cases, a barrier coating may provide sufficient protection.

When shoring is used, the effect of shoring removal on the available strength of the slab should be considered. The use of rational engineering analysis is sufficient for this task. Alternately, it is conservative to consider the slab to carry all dead and *superimposed loads* over the full unshored span.

E2 Design of Deck to Carry Dead and Live Loads

On short *deck* spans, when the *deck* is protected against deterioration, it is possible for the *deck* to carry all dead and live loads without relying upon a structural concrete slab. In this instance, the concrete can be considered to be a non-structural topping. However, for serviceability reasons, it is recommended that temperature and shrinkage control be considered, using either the temperature and shrinkage reinforcement requirements of ACI 318, or other methods such as fibers.

SECTION F - Design of Composite Steel Deck-Slabs for In-Service Conditions

Composite steel floor deck includes *deck* which is capable of developing composite action with the concrete slab by mechanical means.

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,
(d) holes in the *deck* web or flange which allow for bonding of the *deck* to the concrete.

While chemical bond can contribute to the adherence of the concrete to the *deck*, *deck* which develops composite action solely by chemical bond without mechanical bond does not meet this requirement.

F1.1 Load Determination

Load Tables

By using the reference analysis techniques or test results, the *deck* manufacturer determines the live loads that can be applied to the *composite steel 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 (gross) area of the cross section may be used.

Most published *superimposed 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.

Load tables are generally calculated by using a concrete strength of 3000 psi (21 MPa). *Composite steel deck-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 steel beams or joists.

When shoring is used, the effect of shoring removal on the available strength of the slab should be considered. The use of rational engineering analysis is sufficient for this task.

Permanence of Deck

In normal conditions where the *composite steel floor deck* can reasonably be considered to not deteriorate over time, such as a floor in an enclosed building, the *deck* can be considered to be a permanent component and composite action assumed. In an aggressive environment, such as in an open parking garage, where water and corrosive salts can reach the *steel deck*, or in areas of buildings where excessive moisture or corrosive fumes, such as chlorine or acids can attack the *deck*, it should be considered to be an impermanent form for the fluid concrete, and composite action should not be assumed, and all load should be carried by a specifically designed structural concrete slab. In some cases, a barrier coating may provide sufficient protection.

<u>Calibration of ASD to LRFD</u>

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 steel deck-slabs* and matches the ratio used in the AISC 360 Standard. The effective load factor for this live to dead load ratio is 1.50.

<u>Recommendations for *composite steel floor decks* loaded by repeated wheeled loads</u>

Composite steel 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 steel 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.

Composite steel 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;
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).

F1.2 Concrete

The minimum concrete coverage of 2 inches is for structural design only. *Decks* that have wide top flanges may require thicker concrete cover to reduce the potential for cracking over the *deck* flange. Thicker concrete slabs may be required for fire rated floor assemblies.

F1.3 Steel and Deck Finish

Generally, *composite steel floor deck* is specified as either bare or galvanized steel. 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.

F2.2 Reinforcement Classification

The reinforcement ratio of *composite steel deck-slabs* can be calculated using rational analysis by recognized reinforced concrete theory. One method which complies with the assumptions of ACI 318 and this Standard is as follows:

Slabs with (c/d) less than the balanced condition ratio $(c/d)_b$ are considered under-reinforced, whereas slabs with (c/d) greater than or equal to $(c/d)_b$ are considered over-reinforced.

The compression depth ratio is calculated as:

$$\left(c/d\right) = \frac{A_s F_y}{0.85 f_c' d b \beta_1} \qquad \text{(Eq. C-F2.2-1)}$$

The compression depth ratio for the balanced condition is calculated as:

$$\left(c/d\right)_b = \frac{0.003\left(h - d_d\right)}{\left(\frac{F_y}{E_s} + 0.003\right)d} \qquad \text{(Eq. C-F2.2-2)}$$

Where:

A_s	=	*Steel deck* cross sectional area per unit width of *steel deck*, in.^{2}/ft (mm^2/m)
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	=	$0.85 \quad \text{if } f'_c \leq 4000$ psi (27.58 MPa)
β_1	=	$1.05 - 0.05\left(\frac{f'_c}{1000}\right) \geq 0.65 \quad \text{if } f'_c > 4000$ psi (Eq. C-F2.2-3a) (USCS)
β_1	=	$1.09 - 0.008f'_c \geq 0.65 \quad \text{if } f'_c > 27.58$ MPa (Eq. C-F2.2-3b) (SI)

F2.3 Cracked Section Properties

The method provided here is based upon recognized reinforced concrete theory, and may be used for the calculation of geometric cross section properties for *composite steel floor deck* cross sections with concrete. Alternate methods of rational analysis which consider material properties and cracked cross section properties are also permitted to be used.

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.

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 may be determined from Figure C-F2.3-1 and Equations C-F2.3-1 and C-F2.3-3.

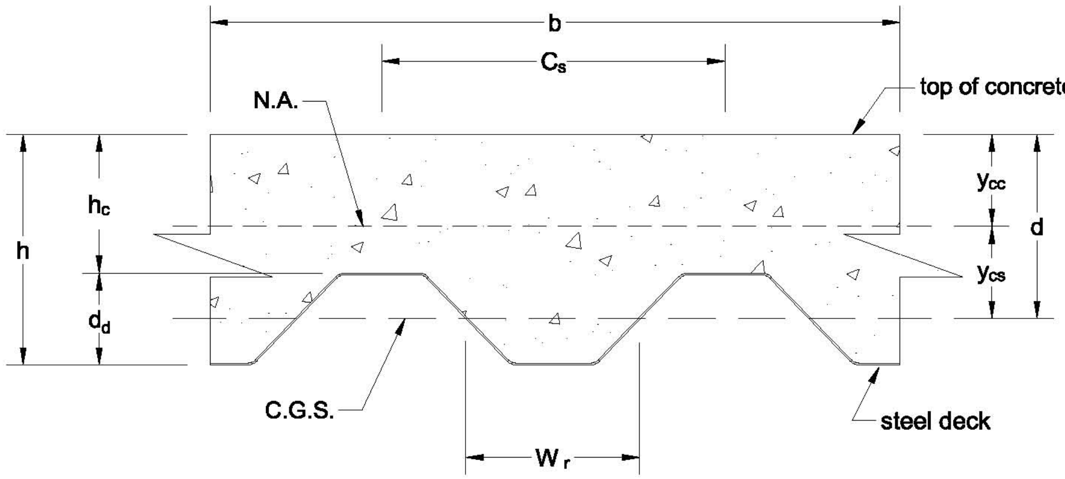


Figure C-F2.3-1 – Composite Section

Note: Figure C-F2.3-1 shows non-cellular *deck*. Section can 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 should 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)

Moment of Inertia of the Cracked Section

For the cracked moment of inertia

$$y_{cc} = d\left[\sqrt{2\rho n + (\rho n)^2} - \rho n\right] \le h_c \qquad \text{(Eq. C-F2.3-1)}$$

Where:

ρ = $\dfrac{A_s}{bd}$
A_s = *Steel deck* cross sectional area per unit width of *steel deck*, 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 = 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^2

= $4700\ (f'_c)^{0.5}$, MPa f'_c in kg/m^2

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} is determined from Equation C-F2.3.1

The cracked moment of inertia transformed to steel, I_{cr} , can be calculated using Equation C-F2.3-2.

$$I_{cr} = \frac{b}{3n} y_{cc}^3 + A_s y_{cs}^2 + I_{sf} \qquad \text{(Eq. C-F2.3-2)}$$

where:

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

Moment of Inertia of the Uncracked Section

For the uncracked moment of inertia

$$y_{cc} = \frac{0.5bh_c^2 + nA_s d + W_r d_d (h - 0.5d_d)\dfrac{b}{C_s}}{bh_c + nA_s + W_r d_d \dfrac{b}{C_s}} \qquad \text{(Eq. C-F2.3-3)}$$

The uncracked moment of inertia transformed to steel, I_u , can be calculated using Equation C-F2.3-4.

$y_{cs} = d - y_{cc}$ where y_{cc} can be determined from Equation C-F2.3-3.

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

(Eq. C-F2.3-4)

Moment of Inertia of the Composite Section

The moment of inertia of the composite section considered effective for deflection computations can be calculated by Equation C-F2.3-5.

$$I_d = \frac{I_u + I_c}{2} \qquad \text{(Eq. C-F2.3-5)}$$

F3 Flexural Resistance of Composite Deck-Slabs

This Standard contains analytical and test-based methods for determining the capacity of *composite steel deck-slabs* and allows for full scale performance testing. All methods are equally acceptable and are the result of years of testing and experience. The *Designer* should not restrict the *deck* manufacturer from choosing any applicable method for determining the flexural resistance of the *composite steel deck-slab*. The limitations of each method are listed within the appropriate section.

The flexural resistance of a *composite steel deck-slab* is the minimum of the ultimate flexural strength (Section F3.1) and the flexural strength governed by *deck* to concrete bond (Section F3.2).

F3.1.1 General Strain Analysis

The purpose of the general strain analysis is to provide a technique for most of those instances when the basic assumptions necessary to use general concrete beam theory for an under-reinforced concrete slab are not met, for cases where it is desired to account for the effects of supplemental reinforcing in the slab, and for cases of over-reinforced sections, as defined in Section F2.2 of the Standard. Several instances may necessitate the use of the general strain compatibility techniques and include the following occurrences:

(1) The entire *steel deck* cross section has not reached yield stress at the instant of the flexural moment capacity. This condition may occur in those slab sections where a larger *deck* depth constitutes a very high percentage of the total slab depth. In this situation the following events might lead to failure:
   a. rupture (tearing) of the bottom steel fibers,
   b. exceeding the maximum concrete compressive force (crushing of concrete),
   c. buckling of the top fiber of the *steel deck* cross section (if in the compression zone).

(2) The centroid of the *steel deck* cross sectional area may not be sufficiently close to the resultant force carried by the steel. This condition may occur when:
   a. the entire *steel deck* section does not yield,
   b. supplementary steel exists in addition to the *steel deck*,
   c. the effective compression plate element widths are less than the full width,
   d. the depth of *steel deck* is large in comparison to the slab depth.

(3) The concrete does not reach the assumed maximum strain of 0.003 inches/inch (0.003 mm/mm). This may take place, for example, if the *steel deck* reaches its rupture stress prior to the concrete reaching its capacity.
(4) The concrete reaches its compressive strength prior to the entire cross section of the *steel deck* reaching its yield. This condition may occur for slabs where the *deck* depth is a very high percentage of the overall slab depth.
(5) The outermost *steel deck* tension fibers may rupture prior to the concrete reaching a strain of 0.003. This condition may occur when the *steel deck* consists of a very high-strength, low-ductility steel.
(6) The *steel deck* slips horizontally with respect to the concrete, but the ultimate failure mode is still that of flexure. This case means that the usual assumption of strain compatibility may not be valid.
(7) The *Designer* wishes to account for the locked-in strains due to casting and shore removal.

The controlling strain most likely to occur in this general analysis is either ε_{C4} or ε_{b4} , as shown in Figure C-F3.1-1. The controlling strain for ε_{C4} should be taken as 0.003. The limiting strain for ε_{b4} depends on the ductility properties of the particular grade of steel. For example, a very ductile steel could easily withstand a limiting ductility strain at the bottom fiber of 50 to 100 or more times the strain corresponding to that of the yield stress. However, a very high-strength steel, such as ASTM A653, Grade 80, may be capable only of a strain of slightly over 0.005 inches per inch (0.005 mm/mm). Thus a limiting tensile strain for ε_B is suggested at 75% of that corresponding to the steel tensile strength strain, if known. If the tensile strength strain is unknown, a strain of about 20-40 times F_y/E_s may be chosen, depending upon the steel's ductility properties. The *Designer* should be careful to select a limiting tensile strain that has an appropriate factor of safety with respect to the strain corresponding to a maximum tensile strength of the steel.

The top fiber of the *steel deck*, ε_{T4} , may also provide a limit as the controlling strain in flexure. This limit would more likely exist for deeper *deck* sections (as a proportion of slab depth) where the top fibers of the *deck* remain in compression. That is, the maximum strain corresponding to local buckling of the top plate elements of the *deck* would provide the proper numerical limit for ε_{T4} .

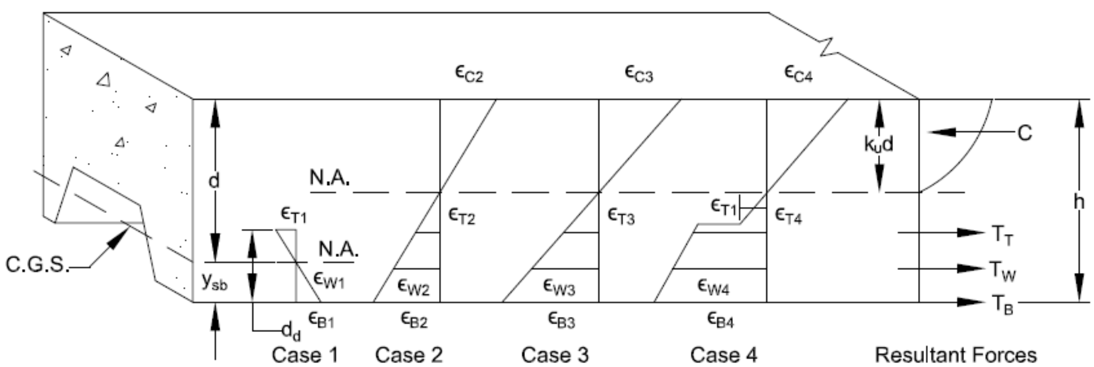
Of the three controlling strains, ε_{b4} , ε_{T4} , and ε_{C4} , the one most likely to control for flexural computations is ε_{C4} , equal to 0.003 inches per inch (0.003 mm/mm). A controlling strain of ε_{B4} may occur for a very high-strength steel with low ductility or for a very deep *deck* section where the depth of *deck*, d_d , is approximately 70% or more of the composite slab depth, h.

Discretion should be exercised when employing general strain analysis to ensure that the proper selection of the controlling strain has been made, particularly for those instances where the *deck* is sufficiently deep to prevent yielding across the entire steel area. Special considerations must be incorporated in the strain analysis if slip should happen to occur prior to ultimate flexural capacity.

Figure C-F3.1-1 is an example of strain diagrams that can be superimposed to obtain the flexural capacity in a general strain analysis. Case 1 in Figure C-F3.1-1 represents strain in the *steel deck* due to casting. This diagram has tensile strains at the top fibers of the *deck* and compressive strains at the bottom fibers, representing the case of a single shore at center span.

The second strain diagram shows strains due to shore removal, assuming that the force exerted on the shore is applied to the composite section. Usually, uncracked transformed moment of inertia values can be used to determine the strains for Case 2. The third case in Figure C-F3.3-1 represents strains due to applied loading. The fourth case is simply an arithmetic sum of the previous cases.

Analysis for flexural capacity is obtained by selecting one of the strains as a limiting strain, say, the bottom *steel deck* strain, ε_{b4} in Figure C-F3.1-1, which may be limited by steel ductility or compatibility of strains across the section.



Note: Figure shows part elevation of slab segment with strain distribution resulting from casting (Case 1), shore removal (Case 2), applied loading (Case 3), and total (Case 4); and resultant forces.

Figure C-F3.1-1
Strain diagrams used to obtain general strain computed flexural capacity of deck-slab elements

If Case 4, ε_{b4} is selected as the limiting strain, moment capacity for live load can be obtained from the Case 3 strains. Correct strain compatibility is achieved when $C = T_T + T_W + T_B$ (see Figure C-F3.1-1). The nominal moment strength, M_n, is obtained as a simple summation of internal moments of the C, T_T, T_W and T_B forces.

In certain cases, the concrete could conceivably slip with respect to the *steel deck* but result in the ultimate failure mode being flexure. For these special cases strain compatibility may not be valid.

Steel decking permitted by the SDI SD Standard and the AISI S100 Standard will usually have adequate ductility for yielding to occur over the entire cross section, except possibly *decking* made from Grade 80 steel. Significant cracking in the steel (not the coating) because of the cold-forming operation is an indication that the *deck* may not have adequate ductility, and appropriate material tests should be made to determine whether the steel is suitable for the intended application. *Steel decks* with extensive cracks should be rejected.

Sections containing supplementary reinforcing steel in addition to the *steel decking* may be analyzed either considering or ignoring the supplemental reinforcing steel.

Simplified Solution Where No Part of the Steel Deck Has Yielded and the Slab Neutral Axis is Above the Top Flange of the Deck ($c \le h_c$)

For *composite steel deck-slabs* where no part of the *steel deck* has yielded and the slab neutral axis is above the top flange of the *deck* ($c \le h_c$), and the strength contribution of supplemental reinforcing steel is not considered, Equation C-F3.1-1 can be used to calculate the nominal resisting moment.

$$M_n = 0.85 f_c' b c \beta_1 (d - c\beta_1 / 2) \qquad \text{(Eq. C-F3.1-1)}$$

Where:

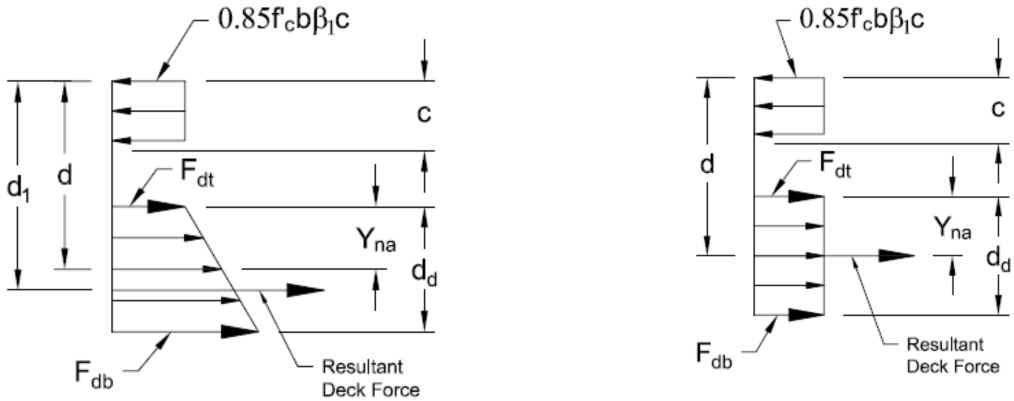
$$c = d\left\{\sqrt{\rho m + \left(\frac{\rho m}{2}\right)^2} - \frac{\rho m}{2}\right\} \qquad \text{(Eq. C-F3.1-2)}$$

$$\rho = \frac{A_s}{bd} \qquad \text{(Eq. C-F3.1-3)}$$

$$m = \frac{E_s \varepsilon_{cu}}{0.85 f_c' \beta_1} \qquad \text{(Eq. C-F3.1-4)}$$

E_s = modulus of elasticity of *steel deck*
= 29,500,000 psi (203,000 MPa)
ε_{cu} = 0.003

Equation C-F3.1-2 is based on simplified assumptions of uniform *deck* stresses and the resultant *deck* force along the *deck* centroid. This assumption is conservative, as shown in Figure C-F3.1-2.



Resultant at Centroid of Force

Assumption of Resultant
at Centroid of Deck Section

Figure C-F3.1-2 – Centroid of Force in Deck Section

Simplified Method for Under-reinforced Sections When the Entire Deck Section Yields

For *composite steel deck-slabs* where the entire *steel deck* section has yielded, and the strength contribution of supplemental reinforcing steel is not considered, Equation C-F3.1-5 can be used calculate the nominal resisting moment.

$$M_n = M_{nu} = A_sF_y(d - a/2) \qquad \text{(Eq. C-F3.1-5)}$$

Where:

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

a = developed depth of concrete in the compression zone

= $A_sF_y / (0.85f'_c\, b)$ (Eq. C-F3.1-6)

b = unit width of compression face of *composite steel deck-slab*

d = distance from the extreme compression fiber to the centroid of the *steel deck*

f'_c = concrete strength, ksi

F_y = yield stress of *steel deck*

F3.2 Flexural strength controlled by deck-to-concrete bond

Section F3.2 limits the nominal flexural strength based on either the limiting bond of the concrete to the *steel deck*, or by limiting the slip of the *deck* relative to the concrete slab through the use of shear connectors.

F3.2.1 Prequalified Section Method

The method was first printed in Appendix D of ASCE 3-91 and was primarily developed by Luttrell with ASCE 3 Committee contributions. A concise history of the method is presented in Luttrell's 2000 paper, Composite Deck Formulations. The ASCE 3-91 Standard included three general types of *embossments*, and strength could be calculated using three constants plus the *embossment* depth measured from the *deck's* web. The aim was to establish minimum *embossments* to develop the yield moment, M_y , of the *deck* profile. The final form of the nominal resisting moment, M_{no} , was:

$$M_{no} = KM_y \qquad \text{Where } K = K_3 / (K_1 + K_3)$$

The constants, K_i, are functions of the profile parameters such as *pitch*, *panel* cover width, and depth/thickness ratio (slenderness) plus *embossment* parameters such as number, spacing, orientation and depth. K can be less than one but KM_y could not exceed M_u, the ultimate strength of the slab. The latest SDI edition requires that the useable K cannot exceed one unless studs are present. The ASCE method is test based and semi empirical because it uses mechanics, traditional concrete slab design, plus rational engineering. Examples of rational engineering are: more bearing area is going to increase developed strength, greater F_y and thickness are going to increase the demand on a constant bearing area pattern, and top flat width and longitudinal *stifeners* can impact results as transverse thrust is developed while resisting end slippage. The SDI SD Standard, in Section A3.1.1.c, requires a maximum useable F_y = 50 ksi and sufficient ductility to form the required profile shape and *embossment* depth without splitting.

Much of the testing had shear spans (area of constant shear near a support) between 0.2L and 0.4L, which covers most flexural design practical cases. This is associated with a relatively constant demand on the *embossments* while developing yielding. Some parameters are not explicitly present in the equations but are built into the applicable range of parameters in the test program.

The SDI has traditionally limited the allowable uniform load (ASD) to 400 psf. Using the safety factor of 1.75, this equates to a nominal uniform load of 700 psf. Therefore,

$$M_{n\,max} = \frac{w_n(\alpha L)^2}{8} = \frac{700(\alpha L)^2}{8} = 87.5(\alpha L)^2 \qquad \text{(Eq. C-F3.2-1)}$$

The same limit, applied to shear

$$V_{n\,max} = \frac{w_n(\alpha L)}{2} = \frac{700(\alpha L)}{2} = 350(\alpha L) \qquad \text{(Eq. C-F3.2-2)}$$

Where: α = 1 when L is in feet, and 0.0348 when L is in meters.

These limits on moment and shear can be applied to uniform loads, and other than uniform loads (including concentrated loads).

The ASCE 3-91 Standard's method was used to evaluate performance in several test programs at West Virginia University (WVU) during the 90's. Many were proprietary for particular manufacturers.

Luttrell simplified the ASCE method in 2000 and this revision was first included in the SDI C-2011 Standard. The denominator was consolidated into one constant, K_1 , with K_1 unique to the *embossment* type. The revisions were compared with a significant body of test data. The average ratio, P_m , (P_{test} / P_{theory}) was good and the scatter was acceptable for composite slab tests. The test method does not always fully simulate benefits present in floors such as repeating *panels* and end restraint of adjacent slab spans or construction perimeter details. K_3 = 1.4 does address repeating *panels*.

This method does not significantly change the results presented in the SDI Composite Deck Design Handbook, 2nd Edition, which was based on tests at WVU and Virginia Polytechnic Institute and State University (VPI). The exception of limiting strength to that of 18 *gage* thickness is removed. The method provides a means to calculate the required *embossments* for manufacturers that did not participate in the earlier research.

The limit on the *deck* web angle is illustrated by Figure C-F3.2-1.

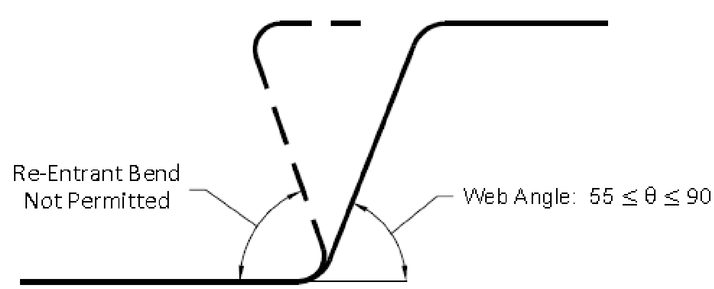


Figure C-F3.2-1 – Limit on Deck Web Angle for Prequalified Method

Limits on base metal thickness, *deck* depth, *rib pitch*, and concrete cover reflect the limits of the tests that were used to develop the *embossment* factors.

Embossments are permitted to protrude into or away from the concrete without adversely affecting the shear-bond strength as discussed in Tremblay et al. (2002). *Embossments* protruding away from the concrete are typically encountered when using inverted, 1 1/2 inch composite *deck*.

This method is applicable if steel headed stud anchors (studs) are not present on the beam flange or open web steel joist chord supporting the *composite steel floor deck*, or if steel headed stud anchors are present in any quantity.

F3.2.2 Shear Bond Method

Additional information on design using shear bond testing can be found in the Commentary to the SDI T-CD Standard.

The strength relationship given by the equations throughout this section of the Standard are based on the concept of uniformly loaded one-way slab action in the direction of the *steel deck corrugations*. This concept is valid for most normal design conditions, except possibly for floor slabs subjected to heavy concentrated loads. For slabs subjected to concentrated loads, special consideration must be given to the design shear span.

This method is applicable if steel headed stud anchors (studs) are not present on the beam flange or open web steel joist chord supporting the *composite steel floor deck*, or if steel headed stud anchors are present in any quantity.

F3.2.3 End Anchorage by Steel Headed Stud Anchors Welded Through Deck

Steel headed stud anchors (studs) are required to develop the ultimate strength of a *composite steel deck-slab* unless testing shows otherwise. SDI SD, Section F3.2.3 provides the design method, which is consistent with the ultimate strength method for reinforced concrete slabs in ACI 318. The SDI procedure with studs has been confirmed by testing at Virginia Polytechnic Institute and at West Virginia University. An example is Widjaja (1996). The ultimate strength resistance, M_{nu} , often is not required in service. The resisting moment, M_{no} , or yield moment, M_y , typically are more than adequate when determining the available strength. Studs are not required in such circumstance and the impact of composite slab action on studs can be neglected.

SDI SD Section F3.2.1 provides the design method where a combination of concrete bond, *deck* profile and indentations resist horizontal shear and develops as much tensile force as possible in the *deck*. This is sometimes called interfacial resistance. Normally, this combination is able to yield the bottom flange of the *deck* and a portion of the webs but will not yield the full cross sectional area of the *deck*. The SDI SD Section F3.2.3 design method uses studs to yield the remainder of the cross section and develop the ultimate moment, and this rationalization is the basis of Equation F3.2-15, which estimates the required force that is delivered to the studs. The studs efficiently resist end slippage and push up over interior support beams in response to slab compression. Equation F3.2-14 estimates the required number of studs. The nominal strength of studs is determined in accordance with AISC 360. Calculation using Equation F3.2-14 is less accurate than the calculation of the slab's ultimate strength and is a conservative estimate of the required number. See AISI S310 or SDI-DDM04 for the diaphragm resistance of *composite steel deck-slabs* and the contribution of studs, which differs from resistance at slabs. Additional studs can be required over lateral force resisting systems.

All composite slab action requires a means to transmit horizontal shear in the concrete and resist horizontal slippage at slab ends. Slippage manifests over supports as concrete longitudinally slides over the *deck's* end as flexural cracking increases in the slab and the nominal moment capacity is approached. With no slippage the ideal flexural strain distribution is linear in the slab, which includes the *deck*. With slippage the strain distribution across the slab is not ideally linear and degrees of partial composite action can exist up to the point of failure. In testing, horizontal shear transfers the maximum moment's compression in the concrete along the shear span between the point of maximum moment and the slab's end, which commonly has no extraneous restraint. When studs are present, additional restraint exists and the ultimate moment can be developed. The horizontal shear resistance combination develops the equivalent maximum tension in the *deck* at the point of maximum moment. At ultimate moment the *deck* is fully yielded. Most testing includes a shear span between 1.5 and 4 ft and often produces constant vertical shear along the shear span.

The studs typically also develop composite beam action and the beam's forces on the stud are perpendicular to the composite *deck* forces. The vectors that load the stud are not necessarily at the same eccentricity relative to the beam and the placement restrictions imply that the tensile force in the *deck* is being anchored near the bottom of the stud. The interaction of forces is more forgiving than simple vector addition but this is an acceptable design method. Using Equation F3.2-15, a 3x12x.0478 inch, 40 ksi composite *deck* requires an approximate F_T = 16.8 kips/ ft width. With one stud having a nominal capacity of 21.9 kips/stud in concrete with f'_c = 3ksi, the useable composite beam capacity is greater than 14 kips/ stud or 64 percent of the normal nominal capacity.

Considering the conservative nature of this assumption, the impact of stud placement within the *corrugation*, and the minimal loss of full composite beam moment capacity versus partial composite beam capacity, this reduction might be negligible. However, the design engineer should consider this impact and the impact of stud placement within the *corrugation* when determining composite beam action. A rational consideration is that opposing action of adjacent slab spans will minimize the need for slip resistance, and the concern is most applicable at free ends such as large holes or building perimeters. However, the support anchorage of welds or studs is still required through the *deck* for safety and to develop tension, and the stud placement requirements must be followed.

The residual stresses from steel production or roll-forming that are locked into the *deck*, and the form stresses due to un-shored concrete placement are balanced before the concrete reaches its full compressive strength, i.e. tension = compression in the *deck*. At ultimate moment with full yielding across the *deck*, the net impact on strength is zero and it is rational to allow the composite slab to resist all loads including the slab dead load that are present after the required concrete compressive stress is reached.

Figures C-F3.2-2, C-F3.2-3, and C-F3.2-4 illustrate proper stud installation.

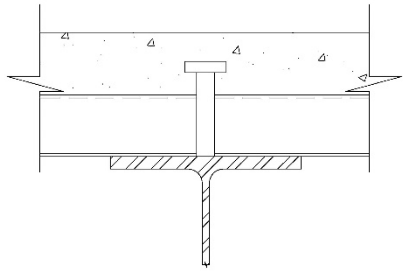


Figure C-F3.2-2
Studs installed not at a deck end or butt joint are installed through the deck.

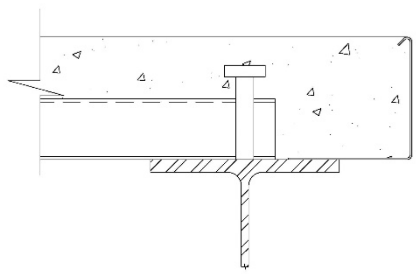


Figure F3.2-3
Studs at a deck end are required to anchor the deck and must be installed through the deck.

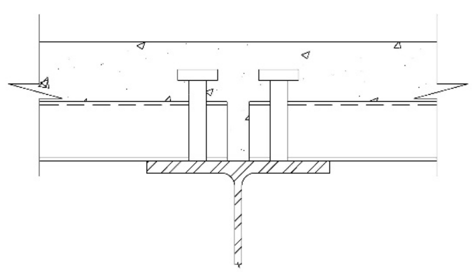


Figure F3.2-4
Studs at a deck butt joint must be installed individually through deck ends and not centered on the butt joint. The full number of studs anchoring the deck, N_s, must be installed on each end of the deck at both sides of the butt joint.

Minimum edge distance for studs, measured from the center of the stud to the end of the *deck*, are as follows:

3/8 inch diameter stud	l_e min = 0.75 inches
1/2 inch diameter stud	l_e min = 0.94 inches
5/8 inch diameter stud	l_e min = 1.12 inches
3/4 inch diameter stud	l_e min = 1.31 inches
7/8 inch diameter stud	l_e min = 1.50 inches

Designers should be aware that while studs up to 7/8 inch in diameter may be used to anchor *deck* for the purposes of this section, AISC 360 limits the design of composite steel beams with *deck* to a maximum stud diameter of 3/4 inch.

Limits on base metal thickness, *deck* depth, *rib pitch*, and concrete cover reflect the limits of the tests that were used to validate the anchorage requirement. It may be possible to exceed these limits, based upon testing, rational engineering analysis, or a combination thereof.

F4 One-Way Shear Strength

One-way shear will rarely, if ever, control the design of a *composite steel deck-slab*. The method in the Standard is the addition of the concrete capacity determined using ACI 318 criteria and the shear capacity of the *deck*.

F5 Two-Way Shear Strength

Two-way (or punching) shear should be checked when there is a heavy concentrated load applied to a small area of the slab. This check is the ACI 318 criteria for 2-way shear, and conservatively neglects the contribution of the *steel deck* to punching.

F6 Lateral Distribution of Concentrated Loads

Concentrated loads on *composite steel deck-slabs* are permitted to be laterally distributed perpendicular to the *deck ribs* by methods of rational analysis. One method of rational analysis, which is found in the SDI-FDDM02, is as follows:

Concentrated loads are permitted to be distributed laterally (perpendicular to the *ribs* of the *deck*) over an effective width, b_e. A uniform load distribution over an effective width, b_e , can be calculated using the following method.

Figures C-F6-1 and C-F6-2 illustrate the dimensions associated with this method.

The concrete above the top of *steel deck* should be designed as a structural reinforced concrete or plain concrete one-way slab transverse to the *deck ribs*, in accordance with ACI 318 Chapter 7, 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 should be applied to the weak axis moment.

$$b_m = b_2 + 2\ t_c + 2\ t_t \quad \text{(Eq. C-F6-1)}$$

$$b_e = b_m + (2)(1\text{-}x/L)x \leq 106.8(t_c/h) \quad \text{(Eq. C-F6-2)}$$
for single span bending

$$b_e = b_m + (4/3)(1\text{-}x/L)x \leq 106.8\ (t_c/h) \quad \text{(Eq. C-F6-3)}$$
for continuous span bending when reinforcing steel is provided in the concrete to develop negative bending

$$b_e = b_m + (1\text{-}x/L)x \leq 106.8\ (t_c/h) \quad \text{(Eq. C-F6-4)}$$
for shear

$$W = L/2 + b_3 \leq L \quad \text{(Eq. C-F6-5)}$$

$$M_{wa} = 12\ P\ b_e\ /\ (15W) \text{ in.-lb per foot} \quad \text{(Eq. C-F6-6a) (USCS)}$$
$$= P\ b_e\ /\ (15\ W) \quad \text{N-mm per mm} \quad \text{(Eq. C-F6-6b) (SI)}$$

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 steel 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)

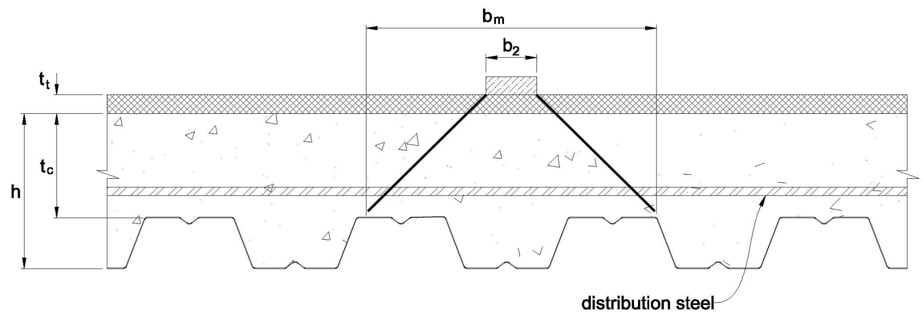


Figure C-F6-1 – Lateral Distribution

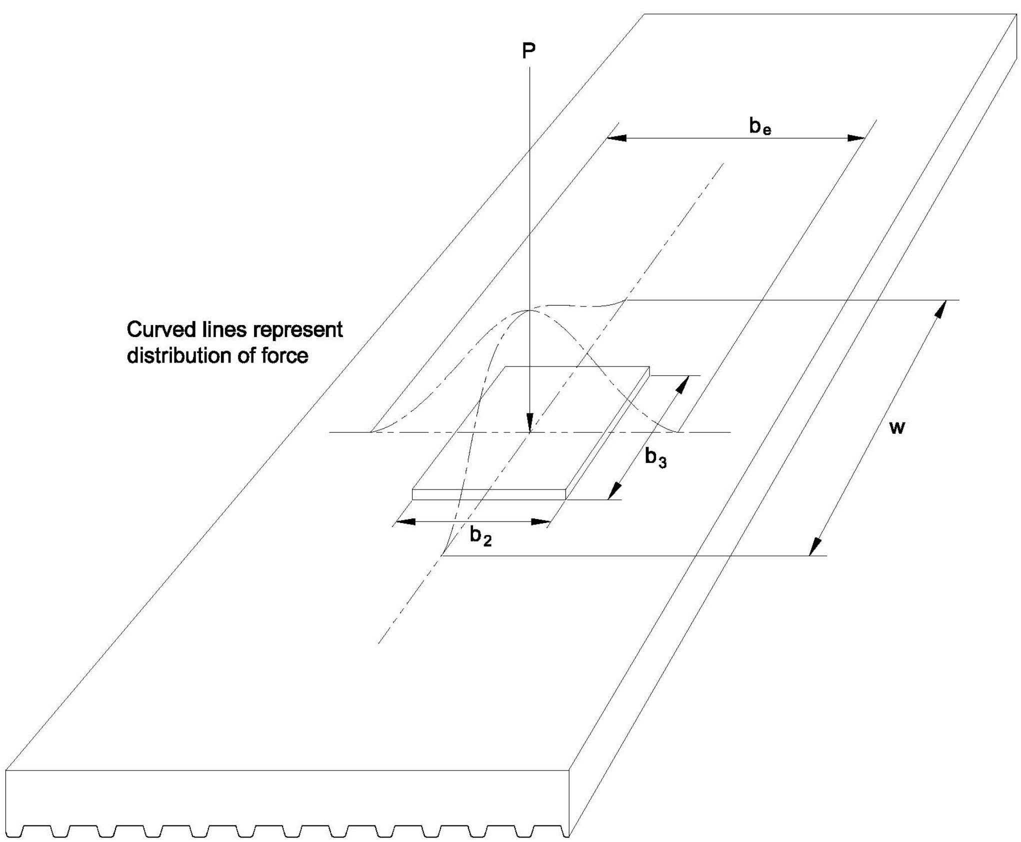


Figure C-F6-2 – Lateral Distribution

When transverse steel reinforcing is included in the slab, this reinforcing can be used as part of a reinforced concrete beam, to resist the weak axis bending moment. When no transverse steel reinforcing is provided, the flexural resistance should be taken as that of the plain concrete section above the top of the *deck ribs*, and the section designed as a plain concrete flexural member as permitted by ACI 318. Any contribution from the *steel deck* should be neglected. The use of fibers for structural reinforcing transverse to the *deck* is outside the scope of the Standard.

F7 Negative Moments over Supports

Composite steel floor deck does not function as compression reinforcing steel in areas of negative moment, such as over supports and at cantilevers. 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.

The coefficient method of ACI 318 Section 6.5 is considered an acceptable analysis method within the limitations inherent within that method.

F8 Cantilevered Slabs

See commentary for Section F7.

F9 Reinforcement for Temperature and Shrinkage

General

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.

Designers should be aware that the majority of slab cracks are not the result of temperature or shrinkage strains, but are the result of the following:

1. Negative moments over supports, resulting from applied loads on the *composite steel deck-slab*. These develop most commonly when the slab is loaded with construction materials before the concrete attains full strength. The SDI-FDDM02 shows one method for calculating the negative moment concrete strain.
2. Operation of vehicles, forklifts or other heavily loaded wheeled equipment on the finished slab.

Provisions for the control of temperature and shrinkage strains will most likely not be adequate to prevent or control cracks resulting from these situations.

Continuous Reinforcement

The prescription of 0.00075 times the area of the concrete above the *deck* in Section G9.1 for continuous reinforcing for temperature and shrinkage is half of the amount of reinforcing prescribed by ACI 318 in Section 24.4.3. This reduced amount of reinforcing considers the contribution of the *steel deck* to the total reinforcement of the *composite steel deck-slab* for shrinkage purposes. This prescription was first codified in ASCE 3-84, based upon engineering judgment. No specific testing or rational analysis was used in developing this prescription.

Shrinkage

Concrete floor slabs employing Portland cement will start to experience a reduction in volume as soon as they are placed, due to plastic shrinkage and drying shrinkage. Commentary Section R24.4.2 of ACI 318 states: "Topping slabs also experience tension due to restraint of differential shrinkage between the topping and the precast elements or metal *deck* (which has zero shrinkage) that should be considered in reinforcing the slab." Where shrinkage is restrained, cracking will occur in the floor. The use of the appropriate methods for shrinkage strain control is intended to result in a larger number of small cracks in lieu of a fewer number of larger cracks.

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 associated cracking include cement type, aggregate type and gradation, water content, water/cement ratio, and the use of reinforcing steel or other admixtures, which may include distributed steel or synthetic fibers.

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).

Temperature

The coefficient of thermal expansion of portland cement concrete (0.00055 inches per inch per 100 degrees) and steel (0.00065 inches per inch per 100 degrees) are close enough that significant thermal strains are unlikely to develop in the concrete slab, and tension strains that develop in the *composite steel deck-slab* due to restraint of the framing will be resisted primarily by the *steel deck*.

Discontinuous Fibers

Discontinuous fibers are an admixture added to the concrete mix to improve performance. Both steel and synthetic fibers have been successfully used for control of strains resulting from temperature effects and shrinkage in the concrete. The 2006, 2011 and 2017 SDI-C Standards provided a prescription of a minimum dosage of 4 pounds per cubic yard of concrete for synthetic macrofibers and 25 pounds per cubic yard for steel fibers. This has been proven in practice to provide acceptable performance. Most adopted building codes provide for alternate materials and methods of construction, therefore the use of fibers outside the minimum prescribed dosages may be possible in specific instances.

Methods for demonstrating performance may include residual strength beam tests conducted in accordance with ASTM C1609/C1609M. There is no complete concensus on how to apply the results of this test, or any other test. *Designers* are directed to product manufacturers for additional guidance.

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*.

Flexural Cracks

Because *composite steel 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. 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 steel deck-slab* for negative moments over supports (both beams and girders) and providing appropriate reinforcing steel at these supports.

F10 Deflection

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 Section F2.3 of this Commentary or SDI-FDDM02.

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.

F10.2 Time Dependent Deflections

Concrete shrinkage and creep cause time-dependent deflections in addition to the elastic deflections that occur when loads are first placed on the *composite steel deck-slab*. Such deflections are influenced by temperature, humidity, curing conditions, age at time of loading, amount of compression reinforcement (if any), and magnitude of the sustained load.

Information on creep deflections of *composite steel deck-slab*s is limited. This method is similar to the procedure for reinforced concrete slabs. The additional load factors given in this section are derived from ACI 318 and are considered conservative. The deflection calculated in accordance with this section is the additional time-dependent deflection due to the dead load and those portions of other loads that will be sustained for a sufficient period to cause significant time-dependent deflections.

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.

The provisions of Section 10.2.3 are similar to those in the footnotes to ACI 318, Table 24.2.2.

SECTION G - Diaphragms

G1 Diaphragm Design

For diaphragm design, the span measured from center-to-center of supports should be used.

AISI S310 is based upon the analytical method presented in SDI-DDM01 and expanded upon in DDM02 and DDM03. AISI S310 permits the use of other models for determining diaphragm strength and stiffness, and permits the use of alternate fasteners. Refer to AISI S310 and the Commentary to AISI S310 for additional information.

SDI-DDM04 contains diaphragm load tables that comply with AISI S310.

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.

SECTION H - Design of Connections

H2 Welding

H2.1 Controlling Standard

In order to comply with AWS D1.3 requirements, welders must demonstrate the ability to produce satisfactory welds using a procedure in accordance with AWS D1.3.

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

H2.2 Weld Washers

AWS D1.3 and AISI S100 do not require the use of weld washers for welding sheet steel 0.028 inches (0.71 mm) and thicker. *Deck* meeting SDI recommended 22 *gage* (0.0295 inch design thickness) or thicker therefore do not require weld washers. Using weld washers on thicker *deck* may actually result in a lower strength weld.

H2.3 Welding without Weld Washers

For arc spot welds, the SDI recommends specifying the visible diameter of the weld (i.e. 5/8 inch or 3/4 inch visible diameter arc spot weld) for ease of installation and inspection. Increasing the minimum weld visible diameter for greater thickness of *deck* may be required to develop the minimum 3/8 inch effective weld diameter required by AISI S100.

H2.5 Bearing Surfaces

AWS D1.3, Section 7.3.2 states; "Close Contact. The parts to be joined by welding shall be brought into close contact to facilitate complete fusion between them."

H3 Mechanical Fasteners

Mechanical fasteners (screws, power-actuated 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 dependent upon both *deck* and support thickness.

SECTION I - Manufacturing Tolerance

I1 Tolerance of Delivered Material

No restriction should be placed on over length *panels* in *end lapped* installations because there is no adverse consequence in this application.

It is not intended that the 90% of design depth be understood as meaning that every *embossment* must comply with this limit. The average of a representative sample of *embossments* on any one *panel* meeting this requirement should be considered to be adequate.

SECTION J - Installation

J1 Deck Fastening and Attachment

J1.1 Deck Support Attachment

When using arc spot or arc seam welds for attachment of the *deck* to the supporting structure, the requirements of AWS D1.3 should be followed for weld quality. Section 8.1.1.4 of AWS D1.3 states; "Undercut. The cumulative length of undercut shall be no longer than L/8, where L is the specified length of the weld or in the case of arc spot welds, the circumference, provided fusion exists between the weld metal and the base metal. Depth of undercut is not a subject of inspection and need not be measured. Melt-through that results in a hole is unacceptable."

The SDI *interprets* that melt-through that occurs within the L/8 undercut region is acceptable, and that good fusion at 7/8ths of the weld perimeter meets the intent of the AWS requirement.

A limited number of holes in the *deck* due to welds missing the support structure is considered acceptable, as long as the missed weld is re-welded in the proper position. *Designers* and inspectors should use rational judgment when determining what constitutes excessive holes in the *deck*.

When the *side-lap* 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 *side-lap* is a *nestable side-lap* a single fastener through both sheets of *steel deck* is acceptable to secure both sheets.

The 12-inch support attachment spacing requirement in hurricane prone regions is intended to limit prying on fasteners due to *deck* cross bending when the uplift demand on the fasteners is high.

J1.2 Deck Side-Lap Fastening

The *side-lap* spacing is a minimum. Service loads or diaphragm design may require closer spacing or larger *side-lap* welds. Good metal-to-metal contact is necessary for a good *side-lap* weld.

The SDI does not recommend fillet welded or arc spot welded *side-laps* for *deck* that is thinner than 0.0358 inch design thickness (20 *gage*) due to difficulty in welding thinner material. Arc top seam welds can successfully be made in 0.0295 inch (22 *gage*) or heavier material due to the weld configuration.

J1.3 Deck Perimeter Attachment Along Edges Between Supports

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 Number 14 screw may be required for thicker and harder steels.

Attachment of the *deck* perimeter is important when the *floor* or *roof deck* is being used as a diaphragm, and the shear is being carried out of the diaphragm at the perimeter. The attachment called for in the section should not be considered as being adequate for all diaphragm applications, and the *Designer* is referred to the SDI-DDM04 and AISI S310 for additional information.

J1.5 Bearing Contact

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. Some minor peening down of the *deck* should be permitted to remove gaps. Inherent tolerances of the supporting structure should be considered by the *Designer*.

J3 Cleaning Prior to Concrete Placement

Debris to be removed includes, but is not limited to, welding rods, excess fasteners, trash, and organic materials.

Stud ferrules which are broken free from the stud do not need to be removed. The ceramic ferrules used for stud welding are made from a fireclay mixture that is compressed with organic binders. The ferrules are totally inert and have no ingredients that would react with or be detrimental to the poured concrete. After firing, the ferrules are similar to standard duty refractory bricks and have a density of 140-150 pounds per cu. ft. The compressive strength of the fired ferrules is approximately 10,000 psi. The ferrule material meets the requirements of the ASTM C-330 Specification for the aggregate material. The broken ferrules on the *deck* should not be collected into clumps or piles. If they are left scattered and loose on the *decking* they should become adequately wetted and mixed with the concrete as it is poured. They will simply be added to the aggregate in the concrete mix as the concrete is placed. The broken ferrules will not interfere with the placement or strength of the concrete.

J4 Reinforcing Steel and Imbedded Items

J4.2 Conduits

The limitations for conduits imbedded in *composite steel deck-slabs* are taken from ASCE 9.

J4.3 Aluminum Items

Aluminum in contact with concrete will release hydrogen gas and unless properly isolated from the *steel deck* and concrete, will cause galvanic corrosion of the *steel deck*.

J5 Temporary Shoring

When selecting *deck*, the *Designer* should pay particular attention to *deck* spans which require temporary shoring as shoring costs may offset any savings created by lighter *deck gages*. *Designers* should consult with a *deck* manufacturer for assistance with this. Particular attention needs to be paid to areas where the *deck* must be installed in a single span condition due to building geometry.

The design of the shoring is usually the responsibility of the *Owner's Designated Representative for Construction*, and is in all cases not the responsibility of the *deck* manufacturer. The *Designer* should assign responsibility for shoring design in the contract documents.

APPENDIX 1 - Stainless Steel Deck

Stainless *steel deck* is available only from a limited number of manufacturers by special arrangement.

Designers should be aware of potential galvanic action between dissimilar metals, which could include contact between stainless steel and carbon or low alloy steels. The risk of galvanic corrosion is based on many factors, including the type of connection and contact, and the atmospheric exposure.

There is a lack of available performance data for stainless steel *composite steel deck-slabs*. If stainless steels are used in this application, testing to the SDI T-CD Standard is recommended.

APPENDIX 2 - Construction Phase Loads - Strength and Deflection

2.1 Deck Supporting Fluid Concrete

The following construction uniform live loads combined with the weight of the fluid concrete are generally adequate for the following placement and finishing methods:

20 psf	Light Duty:	Concrete transport and placement by hose and concrete finishing using hand tools. This is mainly (but not always) used for smaller floor slab areas.
50 psf	Medium Duty:	Concrete transport and placement by buckets, chutes, or handcarts and concrete finishing using light motorized screeds.
75 psf	Heavy Duty:	Concrete transport and placement by motorized buggies and concrete finishing using large motorized screeds.

The following construction uniform live loads without the weight of the fluid concrete are generally adequate for the following construction operations.

50 psf	Normal Duty:	Concentration of personnel and staging of materials for average construction operations.

Either or both of these construction live loads should be increased when the construction operations dictate. It should be the responsibility of the *Owner's Designated Representative for Construction* to proactively act to ensure that the *deck* is not overloaded by the anticipated construction operations.

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.

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-FDDM02 for additional information.

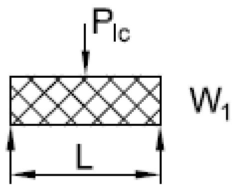
The load factor used for the dead weight of the concrete is 1.6 because of delivering methods and an individual sheet can be subjected to this load. The use of a load factor of 1.4 for construction load in LRFD design is calibrated to provide equivalent design results in ASD design. Refer to the commentary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

Single span *deck* conditions have no redundancy because they are statically determinate, as opposed to multi-span conditions that are statically indeterminate. 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.

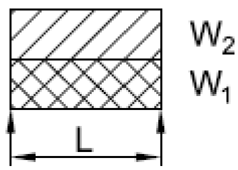
2.1.3 Loading Considerations

The loadings shown below are representative of the sequential loading of fresh concrete on the *deck*.

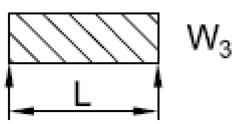
Single Span Condition



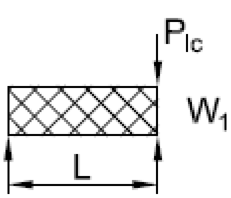
$+M = 0.125\ W_1L^2 + 0.25\ PL$ (Eq. C-A2-1)



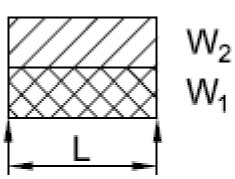
$+M = 0.125\ (W_1 + W_2)\ L^2$ (Eq. C-A2-2)



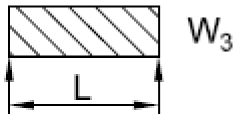
$+M = 0.125\ W_3\ L^2$ (Eq. C-A2-3)



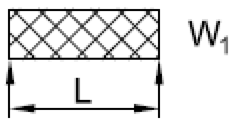
$P_{ext} = 0.5\ W_1\ L + P$ (Eq. C-A2-4)



$P_{ext} = 0.5\ (W_1 + W_2)\ L$ (Eq. C-A2-5)

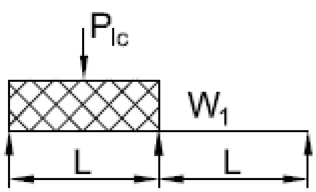


$P_{ext} = 0.5\ W_3\ L$ (Eq. C-A2-6)

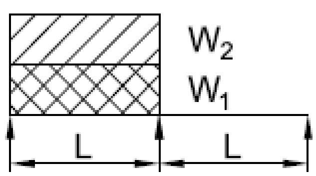


$\Delta = 0.0130\ W_1\ L^4 / E_s I$ (Eq. C-A2-7)

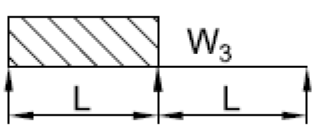
<u>Double Span Condition</u>



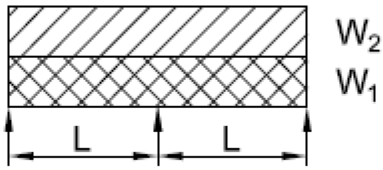
$+M = 0.096\ W_1\ L^2 + 0.203\ PL$ (Eq. C-A2-8)



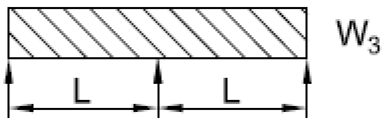
$+M = 0.096\ (W_1 + W_2)\ L^2$ (Eq. C-A2-9)



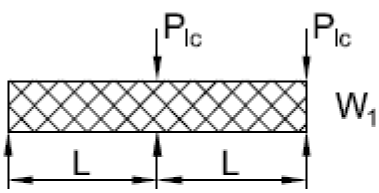
$+M = 0.096\ W_3\ L^2$ (Eq. C-A2-10)



$-M = 0.125\ (W_1 + W_2)\ L^2$ (Eq. C-A2-11)

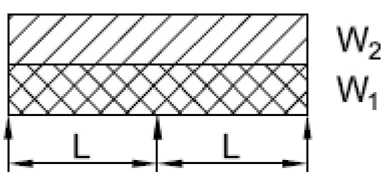


$-M = 0.125\ W_3\ L^2$ (Eq. C-A2-12)



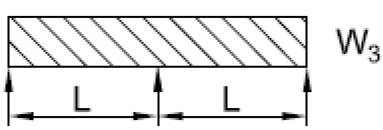
$P_{ext} = 0.375\ W_1\ L + P$ (Eq. C-A2-13)

$P_{int} = 1.25\ W_1\ L + P$ (Eq. C-A2-14)



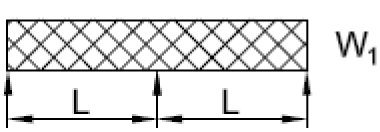
$P_{ext} = 0.375\ (W_1 + W_2)\ L$ (Eq. C-A2-15)

$P_{int} = 1.25\ (W_1 + W_2)\ L$ (Eq. C-A2-16)



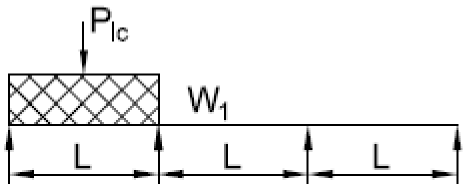
$P_{ext} = 0.375\ W_3\ L$ (Eq. C-A2-17)

$P_{int} = 1.25\ W_3\ L$ (Eq. C-A2-18)

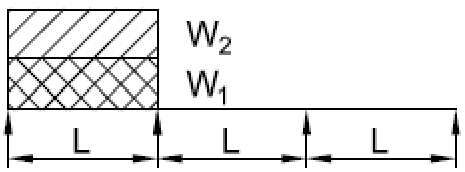


$\Delta = 0.0054\ W_1\ L^4 / E_sI$ (Eq. C-A2-19)

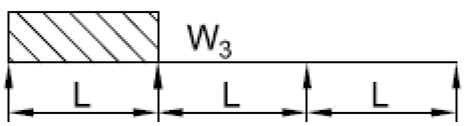
Triple Span Condition



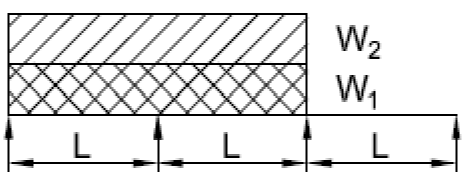
$$+M = 0.094\ W_1\ L^2 + 0.20\ PL \qquad \text{(Eq. C-A2-20)}$$



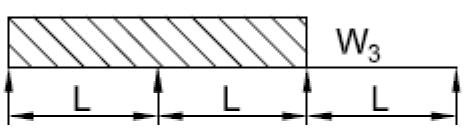
$$+M = 0.094\ (W_1 + W_2)\ L^2 \qquad \text{(Eq. C-A2-21)}$$



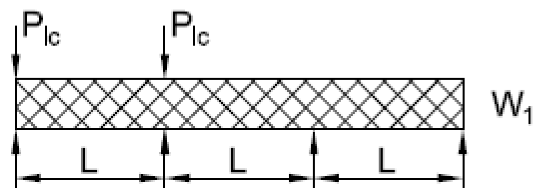
$$+M = 0.094\ W_3\ L^2 \qquad \text{(Eq. C-A2-22)}$$



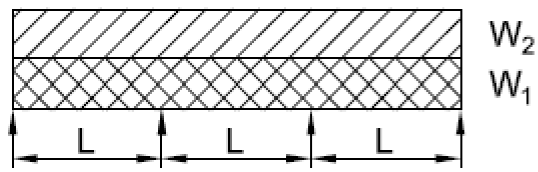
$$-M = 0.117\ (W_1 + W_2)\ L^2 \qquad \text{(Eq. C-A2-23)}$$



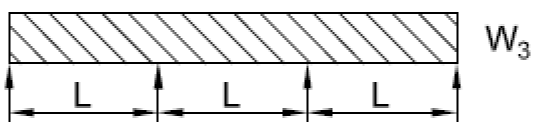
$$-M = 0.117\ W_3\ L^2 \qquad \text{(Eq. C-A2-24)}$$



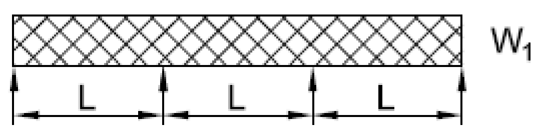
$P_{ext} = 0.40\ W_1\ L + P$ (Eq. C-A2-25)
$P_{int} = 1.1\ W_1\ L + P$ (Eq. C-A2-26)



$P_{ext} = 0.40\ (W_1 + W_2)\ L$ (Eq. C-A2-27)
$P_{int} = 1.1\ (W_1 + W_2)\ L$ (Eq. C-A2-28)



$P_{ext} = 0.40\ W_3\ L$ (Eq. C-A2-29)
$P_{int} = 1.1\ W_3\ L$ (Eq. C-A2-30)



$\Delta = 0.0069\ W_1\ L^4 / E_sI$ (Eq. C-A2-31)

Where:

$P = \alpha_2 P_{LC}$	=	Concentrated construction live load (Eq. C-A2-32)
$W_1 = \alpha_1 W_{dc} + \alpha_3 W_{dd}$	=	Slab dead weight + *deck* dead weight (Eq. C-A2-33)
$W_2 = \alpha_2 W_{lc}$	=	Uniform construction live load (Case 1) (Eq. C-A2-34)
$W_3 = \alpha_3 W_{dd} + \alpha_2 W_{cdl}$	=	*Deck* dead weight + Uniform construction live load (Case 2) (Eq. C-A2-35)
L	=	*Deck* design (clear) span
I	=	*Deck* moment of inertia
E_s	=	*Deck* modulus of elasticity (29500 ksi) (203,000 Mpa)
Δ	=	Maximum *deck* deflection

Dimensional consistency requires consistent units.

Load Factors:

Load Factor	Moment and Reactions		Deflections
	LRFD	ASD	LRFD and ASD
α_1	1.6	1.0	1.0
α_2	1.4	1.0	1.0
α_3	1.2	1.0	1.0

2.1.4 Deck Deflection

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.

The ponding effect of fluid concrete during placement was not addressed directly in ANSI/SDI C-2017 and ANSI/SDI NC-2017. Those Standards referenced SDI-FDDM02, which provides guidance on how additional concrete weight could be calculated. Other standards were reviewed for guidance. The European (EN 1994-1-1) and Australian and New Zealand (AS/NZS 2327) standards require the ponding effect to be accounted for when the calculated central deflection of the *deck* exceeds 1/10 of the slab depth. The standards permit the *Designer* to account for the ponding effects by increasing the nominal concrete thickness over the whole span by 0.7 times Δ , where Δ is the *deck* central deflection. The SDI-SD Standard adopts this approach.

2.2 Deck Not Supporting Fluid Concrete

2.2.1 Supported Spans

The 200 pound per foot of width (2.92 kN per 1 m of width) load is the equivalent of distributing a 300 pound (1.33 kN) worker over an 18 inch (600 mm) width. Experience has shown this to be a conservative distribution.

2.2.2 Cantilever Spans

Equations 2.2-3 and 2.2-4 reflect construction loading on the cantilever with a 300 pound load of a worker distributed over an 18 inch *deck* width, located at the cantilever end. Equations 2.2-3 and 2.2-4 are strength related and the presence of a construction load in the back spans is typically of no consequence, except in the rare instance where web crippling may control.

2.2.3 Loading Considerations

One condition where additional construction live load should be considered is when lightweight insulating concrete (LWIC) is being applied to *roof deck*. The weight of the additional water added to the concrete slurry, which will evaporate, can be substantial and should be incorporated into the design by the *Designer*. The weight of LWIC systems can be obtained from the LWIC manufacturer.