

ANSI / ASCE 3-91
ANSI / ASCE 9-91
ANSI Approved December 11, 1992

ASCE STANDARDS

American Society of Civil Engineers

Standard for the Structural Design of Composite Slabs

ANSI/ASCE 3-91
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Standard Practice for Construction and Inspection of Composite Slabs

ANSI/ASCE 9-91
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ABSTRACT

American Society of Civil Engineers Standard for the Structural Design of Composite Slabs, ASCE Standard Practice for Construction and Inspection of Composite Slabs (ASCE 3-91 and ASCE 9-91 respectively) presents standards for the structural design and testing of composite slabs and for good construction practice and inspection procedures. In addition, commentaries on both standards are included. The "Standard for the Structural Design of Composite Slabs" (ASCE 3-91) and its "Commentary" cover such topics as loads, construction stage, strength design, service load design, test procedures, and test results evaluation. The "Standard Practice for the Construction and Inspection of Composite Slabs" (ASCE 9-91) and its "Commentary" discuss such topics as damage control, connections, concrete placement, shore removal, holes and hole reinforcement. These standards are written in such a form that they may be adopted by reference in a general building code.

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STANDARDS

In April 1980, the Board of Direction approved ASCE Rules for Standards Committees to govern the writing and maintenance of standards developed by the Society. All such standards are developed by a consensus standards process managed by the Management Group F (MGF), Codes and Standards. The consensus process includes balloting by the balanced standards committee made up of Society members and non-members, balloting by the membership of ASCE as a whole and balloting by the public. All standards are updated or reaffirmed by the same process at intervals not exceeding five years.

The following standards have been issued:

ANSI/ASCE 1-88 N-725 Guidelines for Design and Analysis of Nuclear Safety Related Earth Structures
ANSI/ASCE 2-91 Measurement of Oxygen Transfer in Clean Water
ANSI/ASCE 3-91 Standard for the Structural Design of Composite Slabs and ANSI/ASCE 9-91 Standard Practice for the Construction and Inspection of Composite Slabs
ASCE 4-86 Seismic Analysis of Safety-Related Nuclear Structures

Building Code Requirements for Masonry Structures (ACI530-92/ASCE5-92/TMS402-92) and Specifications for Masonry Structures (ACI530.1-92/ASCE6-92/TMS602-92)
Specifications for Masonry Structures (ACI530.1-92/ASCE6-92/TMS602-92)
ANSI/ASCE 7-93 Minimum Design Loads for Buildings and Other Structures
ANSI/ASCE 8-90 Standard Specification for the Design of Cold-Formed Stainless Steel Structural Members
ANSI/ASCE 9-91 listed with ASCE 3-91
ANSI/ASCE 10-90 Design of Latticed Steel Transmission Structures
ANSI/ASCE 11-90 Guideline for Structural Condition Assessment of Existing Buildings
ANSI/ASCE 12-92 Guideline for the Design of Urban Subsurface Drainage
ASCE 13-93 Standard Guidelines for Installation of Urban Subsurface Drainage
ASCE 14-93 Standard Guidelines for Operation and Maintenance of Urban Subsurface Drainage
ASCE 15-93 Standard Practice for Direct Design of Buried Precast Concrete Pipe Using Standard Installations (SIDD)

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FOREWORD

The material presented in this Standard has been prepared in accordance with recognized engineering principles. This Standard should not be used without first securing competent advice with respect to its suitability for any given application. The publication of the material contained herein is not intended as a representation or warranty on the part of the American Society of Civil Engineers, or of any other person named herein, that this information is suitable for any general or particular use or promises freedom

from infringement of any patent or patents. Anyone making use of this information assumes all liability from such use. The appendices contained in this document are intended by the Steel Deck with Concrete Standards Committee to be included with the parent Standard document unless specifically exempted by building code authorities. This standards document is written in such a form that it may be adopted by reference in a general building code.

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ASCE Standard for the Structural Design of Composite Slabs

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NOTATION

a	= depth of equivalent rectangular stress block, $A_s f_y / 0.85 f'_c b$, in.	f'_{ct}	= compressive test cylinder strength of concrete at time of slab testing, psi
A_s	= cross-sectional area of steel deck, or area of negative moment reinforcing steel where used as tension reinforcement, sq. in. per ft. of width	f'_{c1}	= lower usage compressive strength of concrete, used to determine V_{e1} , psi
A'_s	= area of that portion of steel deck which is in compression, sq. in. per ft. of width	f'_{c2}	= laboratory test strength of concrete corresponding to V_{e2} , psi
A''_s	= area of that portion of steel deck which is in tension, sq. in. per ft. of width	h	= nominal out-to-out depth of slab, in.
b	= unit width of slab, 12 in. (305 mm) ¹	h_c	= depth of concrete above top corrugation of steel deck, in.
B	= width of slab, ft.	h_t	= out-to-out depth of slab at failure crack in test specimen, in.
b_d	= width of composite test slab, ft.	I_c	= moment of inertia of composite section based on cracked section, in. ⁴ per ft. of width
B_b	= width of bottom flange measured at intersection of inside tangents, in.	I_d	= moment of inertia of composite section considered effective for deflection computations, in. ⁴ per ft. of width
B_t	= width of top flange measured at intersection of inside tangents, in.	I_D	= moment of inertia of full steel deck section only taken about the composite cracked section neutral axis, in. ⁴ per ft. of width
c_m	= moment coefficient, dependent upon whether the slab is simply supported or continuous, and on distribution of loads	I_e	= effective moment of inertia, in. ⁴ per ft. of width
C	= compressive force on cross section due to flexure, lbs.	I_s	= moment of inertia of steel deck based on effective width [1], in. ⁴ per ft. of width
C_s	= cell spacing, in.	I_{st}	= moment of inertia of steel deck based on full cross sectional deck area, in. ⁴ per ft. of width
d	= effective slab depth, distance from extreme concrete compression fiber to centroidal axis of the full cross section of steel deck, in.	I_u	= moment of inertia of composite section based on uncracked section, in. ⁴ per ft. of width
d_d	= overall depth of steel deck profile, in.	k	= ordinate intercept of reduced experimental shear-bond line
D_w	= developed width of web measured to inside tangent on flanges, including end arcs, in.	k_u	= ratio defining position of neutral axis at failure
e_1	= distance from C-resultant force to top of steel deck, in.	k_1	= ordinate intercept of shear-bond line
e_2	= distance from C-resultant force to mid-height of deck web, in.	K	= bond force transfer property, $K_3 / (K_1 + K_2)$
e_3	= distance from C-resultant force to bottom of steel deck, in.	K_o	= test average for M_o/M_i values
E	= earthquake load perpendicular to slab, psf	K_1	= $[d_d / 7.8]^{0.5}$
E_c	= modulus of elasticity of concrete, psi	K_2	= mechanical bond factor, from Eq. (D-12) or (D-15)
E_s	= modulus of elasticity of steel deck, 29,500,000 psi (203 000 MPa)	K_3	= slab width factor, from Eq. (D-10)
f	= allowable stress, psi	ℓ_e	= length of embossment, in.
f_b	= bending stress for elastic computation, psi	ℓ_f	= length of span or shored span, ft.
f_r	= modulus of rupture of concrete, psi	ℓ'_f	= length of shear span, ft.
f_u	= specified tensile strength of steel, psi	ℓ_i	= length of span or shored span, in.
f_{ut}	= measured tensile strength of steel, according to ASTM A370 [4], psi	ℓ'_i	= length of shear span, in., for uniform load, $\ell'_i = \ell_i / 4$ in.
f_y	= specified or design yield point or yield strength of steel, psi	ℓ''_i	= distance between inflection points in any particular span of a continuous slab, in.
f_{yt}	= measured yield strength of steel, according to ASTM A370 [4], psi	ℓ_{nf}	= length of clear span, ft.
f'_c	= specified compressive strength of concrete, psi		

¹ Unit width of slab in SI units shall be consistent with SI units for other terms.

ℓ_o	= slab overhang at supports, in.	T_w	= component of tensile force resisted by web elements of steel deck in general strain analysis location of resultant force is assumed to be at mid-depth of web elements), lbs. per ft. of width
m	= slope of reduced experimental shear-bond line	V_e	= maximum experimental shear at failure obtained from laboratory tests (not including weight of slab), lbs. per ft. of width
m_1	= slope of shear-bond line	V_{e1}	= shear-bond capacity corresponding to a lower usage compressive strength of concrete, lbs. per ft. of width
M	= moment due to concrete dead load, steel deck load, and construction live load, ft.-lbs. per ft. of width	V_{e2}	= shear-bond capacity from laboratory test compressive strength of concrete, lbs. per ft. of width
M_a	= applied moment, ft.-lbs. per ft. of width	V_n	= nominal shear-bond strength, lbs. per ft. of width
M_{cr}	= calculated bending moment at cracking, ft.-lbs. per ft. of width	V_u	= factored shear force, lbs. per ft. of width
M_{et}	= calculated bending moment at first yield, ft.-lbs. per cell width C_s	W	= wind load perpendicular to slab, psf
M_n	= nominal moment strength, ft.-lbs. per ft. of width	w	= average width of embossment, in.
M_o	= tested moment strength, ft.-lbs. per ft. of width	W_d	= computed uniform dead load, ($W_{dc} + W_{dd} + W_{ds}$), psf
M_s	= bending moment induced by shoring removal, ft.-lbs. per ft. of width	W_{dc}	= concrete dead load including additional weight of concrete due to deck deflection, psf
M_t	= $K(M_{et})/C_s$, bending moment, modified for bond limitations, ft.-lbs. per ft. of width	W_{dd}	= steel deck dead load, psf
M_u	= factored moment, ft.-lbs. per ft. of width	W_{ds}	= superimposed uniform dead load, (additional dead load applied to slab exclusive of W_s), psf
N	= number of cells in test slab width	W_{el}	= superimposed uniform live load, specified by general building code, but not greater than W_{ts} or W_{tf} , psf
n	= modular ratio, E_s/E_c	W_{tc}	= uniform construction live load, 20 psf (1.0 kN/m ²)
N_v	= number of vertical elements in embossment pattern lengths	W_{tf}	= permissible superimposed uniform live load for flexure, psf
N_h	= number of horizontal elements in embossment pattern lengths	W_{ts}	= permissible superimposed uniform live load for shear-bond, psf
P_e	= maximum applied experimental slab load at failure obtained from laboratory tests (includes weight of loading system but not weight of slab), lbs.	W_r	= average rib width, $(C_s - B_t + B_b)/2$, in.
P_{tc}	= concentrated construction live load on a per ft. width of deck, 150 lbs. (2.2 kN/m)	W_{rf}	= roof live loads (see 4.11 of ASCE 7 [3]), snow loads, or rain loads, except ponding, psf
p_h	= height of embossment, in.	W_s	= weight of slab, ($W_{dd} + W_{dc}$), psf
p_s	= embossment intensity factor, $12\ell_e/s$	W_{sw}	= snow load, psf
s	= center-to-center spacing of shear devices, in.; or in Appendix D, the length of repeating embossment pattern, in.	W_u	= uniformly distributed factored load, psf
S	= appropriate effective section modulus for either positive or negative bending, in. ³ /ft. of width	W_{uff}	= permissible superimposed uniform load for flexure, exclusive of W_s , psf
S_c	= section modulus of concrete, in. ³	W_{ufs}	= permissible superimposed uniform load for shear-bond, exclusive of W_s , psf
t	= thickness of steel deck exclusive of coating, in.	X	= abscissa value for shear-bond determination $\rho d/\ell'_i \sqrt{F'_c} = A_s/b\ell'_i \sqrt{F'_c}$
T_B	= component of tensile force resisted by bottom horizontal elements of steel deck in general strain analysis, lbs. per ft. of width	y_{∞}	= distance from neutral axis of composite section to top of slab, in.
T_i	= deck element tension forces with $i = 1$ to 3, lbs.		
T_T	= component of tensile force resisted by top horizontal elements of steel deck in general strain analysis, lbs. per ft. of width		

y_{ca}	= distance from neutral axis of composite section to centroidal axis of steel deck, in.	$\epsilon_{B1}, \epsilon_{B2}, \epsilon_{B3}, \epsilon_{B4}$	= strain in bottom fiber of steel deck for general strain analysis, microinches per in.
y_{sb}	= distance from centroidal axis of steel deck to bottom of steel deck, in.	ϵ_C	= maximum compressive strain in the concrete, taken as 0.003 in./in. (0.003mm/mm)
Y	= ordinate value for shear-bond determination = $V_c/bd\sqrt{f'_c}$	$\epsilon_{C2}, \epsilon_{C3}, \epsilon_{C4}$	= strain at top fiber of concrete for general strain analysis, microinches per in.
α	= coefficient for modifying I_u for deflection calculations	$\epsilon_{T1}, \epsilon_{T2}, \epsilon_{T3}, \epsilon_{T4}$	= strain at top fiber of steel deck for general strain analysis, microinches per in.
β_1	= 0.85 for concrete with $f'_c \leq 4000$ psi (28 MPa) and is reduced at a rate of 0.05 for each 1000 psi (7 MPa) of strength above 4000 psi (28 MPa), but β_1 shall not be taken less than 0.65	$\epsilon_{W1}, \epsilon_{W2}, \epsilon_{W3}, \epsilon_{W4}$	= strain in web sections of steel deck taken at mid-depth of web elements for general strain analysis, microinches per in.
γ	= coefficient for proportion of dead load added upon removal of shore	θ	= web angle, degrees
Δ	= maximum deflection, in.	λ	= multiplier for additional long-time deflection
Δ_i	= iterative total deflection based on Δ to account for ponding or for the additional concrete due to the deflection of the steel during casting, ft.	ρ	= reinforcement ratio of steel deck area to effective concrete area, A_s/bd
Δ_1	= maximum single span deflection, in.	ρ_b	= reinforcement ratio producing balanced strain conditions
Δ_2	= maximum double span deflection, in.	ϕ	= strength reduction factor
Δ_3	= maximum triple span deflection, in.		

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American Society of Civil Engineers Standard for the Structural Design of Composite Slabs

CHAPTER 1 - GENERAL

1.1 - Introduction

1.1.1 - Composite slabs. Composite slab construction is defined as a system comprising of normal weight or lightweight structural concrete placed permanently over cold-formed steel deck in which the steel deck performs the dual role of acting as a form for the concrete during construction and as positive reinforcement for the slab during service.

Steel decks, either shored or unshored, shall provide adequate strength and stiffness to support wet concrete and construction live loads as defined in Section 2.2.3. Composite behavior results when the steel deck has some reliable mechanical means of providing positive interlocking between the deck and the concrete. These connecting means shall provide resistance to separation of the steel deck and the concrete. Where manufacturers' decks provide for variable spacing of the shear devices, this must be accounted for in the design and test procedures.

1.1.2 - Objective and scope. This Standard¹ presents provisions applicable to composite slabs, relating to:

- (a) structural design, and
- (b) testing.

Standard performance tests shall be conducted to evaluate the load-carrying capacity of composite slabs, wherein the steel deck manufacturer shall be responsible for conducting, under supervision of a registered professional engineer and an independent testing agency, slab tests as required in Chapter 3 of this document. Steel deck section properties shall be calculated according to the **AISI Specification for the Design of Cold-Formed Steel Structural Members** [1]² unless otherwise required herein. The **ACI Building Code Requirements for**

Reinforced Concrete, ACI 318 [2] shall be used for the design of composite slabs, unless modified herein. Both shored and unshored installations are considered.

The appendices contained in this design standard shall be included as part of this mandatory document unless specifically exempted by the building code authorities.

1.1.3 - Inch-pound units. Equations appearing in this Standard are compatible with the following Inch-Pound units. However any consistent measurement units such as SI units may be used.

- (a) Force: lbs.
- (b) Span length: in. or ft.
- (c) Shear span length: in. or ft.
- (d) Dimensions of cross section: in.
- (e) Moment: ft.-lbs.

SI units or equations as given in parentheses in this Standard and Appendix A are for information only and are not part of the Standard.

1.2 - Materials

1.2.1 - Steel deck. Grades of steel used in fabricating steel deck units shall be as described in Section A3 of the **AISI Specification for the Design of Cold-Formed Steel Structural Members** [1]. The deck coating shall be appropriate to the environment of the structure and shall be specified by the designer.

1.2.2 - Concrete. Materials for concrete, including admixtures and reinforcing, shall comply with Chapter 3 of **ACI 318**, except as modified herein [2]. Minimum specified compressive strength of concrete, f'_c , shall be 2500 psi (17 MPa) [2]. Structural concrete shall be in accordance with **ACI 318** [2].

¹ For standard practices of construction and inspection, see **ASCE 9 Standard Practice for Construction and Inspection of Composite Slabs** (See REFERENCES in Commentary)

² Numbers in brackets denote **APPLICABLE DOCUMENTS** listed after Chapter 3.

1.3 - Slab limitations

1.3.1 - **Span-to-depth ratios.** Maximum span-to-depth ratios, ℓ_i/h , are

- | | |
|---|----|
| (a) Simply supported span | 22 |
| (b) One end continuous span (see Section 2.3.1.4) | 27 |
| (c) Both ends continuous span (see Section 2.3.1.4) | 32 |

unless deflection calculations are performed to satisfy the particular service conditions.

1.3.2 - **Slab depth and concrete cover.** The minimum slab depth, h , shall be 3-1/2 in. (90 mm). The minimum depth of concrete above the uppermost surface of the steel deck, h_c , shall be 2 in. (50 mm). When additional reinforcement is placed over the steel deck, the minimum cover above the reinforcement shall be 3/4 in. (20 mm).

1.4 - Tolerances and minimum sizes

1.4.1 - **Tolerances.** Tolerances shall be established by the design engineer and approved by the authority having jurisdiction.

1.4.2 - **Dimensions.** Dimensions, including height and spacing of shear transfer devices, shall be as tested in Chapter 3.

CHAPTER 2 - DESIGN CRITERIA

2.1 - Loads

Load shall be as specified by the building code. Loads not covered by the building code shall be in accordance with ASCE 7 [3]. Section 2.2.3 gives the loads for design of the steel deck during the construction stage. Load combinations for the composite section are given in Section 2.3.1.2.

2.2 - Construction stage

2.2.1 - **General.** Steel deck shall be designed to carry concrete dead load, steel deck dead loads, and

construction live load defined in Section 2.2.3 during construction stage before the concrete has set. Shoring shall be used where stresses or deflections in the steel deck exceed their limits.

2.2.2 - **Section properties.** Section properties of steel deck for construction stage, such as cross sectional area, moment of inertia, and other flexural constants, and properties of steel deck elements, shall be determined in accordance with applicable parts of AISI Specification for the Design of Cold-Formed Steel Structural Members [1].

2.2.3 - **Construction live loads.** Steel deck shall be designed to carry the following minimum live loads during the construction stage:

W_{lc} , uniform construction live load = 20 psf (1.0 kN/m²)

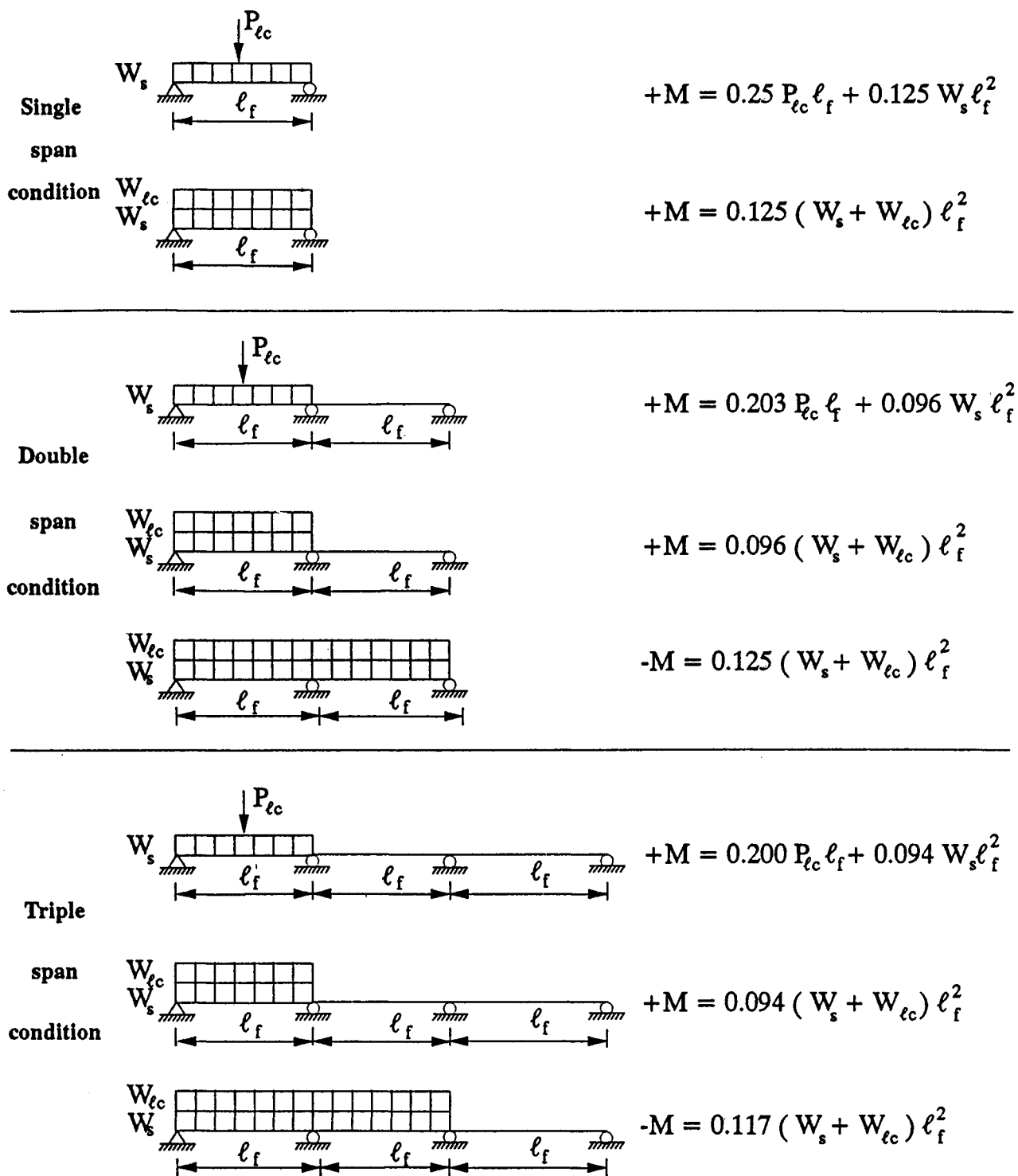
or,

P_{lc} , concentrated construction live load on a per ft. width section of deck = 150 lbs (2.2 kN/m), whichever governs.

2.2.4 - **Allowable stresses.** Calculated stresses shall not exceed allowable stresses given by the AISI Specification for Design of Cold-Formed Steel Structural Members [1].

2.2.5 - **Calculated stresses.** Steel deck stresses for bending, shear, and bearing, caused by the total construction dead and live loads shall be calculated by elastic theory with spans taken as distances between support points. Loads shall be applied in a manner that simulates the sequence of concrete placement as shown in Figure 2.1. For support reaction calculations, loads shall be applied as shown in Figure 2.2. Stresses shall also be calculated for additional loads resulting from the deflection of supporting structural members, (such as concrete ponding), and for other load combinations.

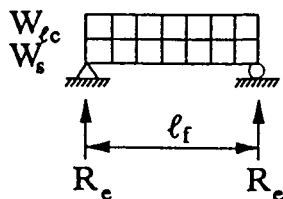
2.2.6 - **Dead load deflection.** Under the weight of the wet concrete, deflection, Δ , over the span of steel deck, shall not exceed lessor of $\ell_i/180$ or 3/4 in. (20 mm). See Figure 2.3.



- Note: 1) Use W_s and W_{lc} on a per ft. length and per ft. width section of deck in equations for calculated moments.
- 2) Additional analysis shall be required for unequal spans

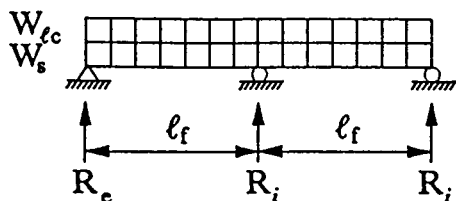
Figure 2.1 - Loading diagrams for moments

Single
span
condition



$$R_e = 0.5 \ell_f (W_{\ell_c} + W_s)$$

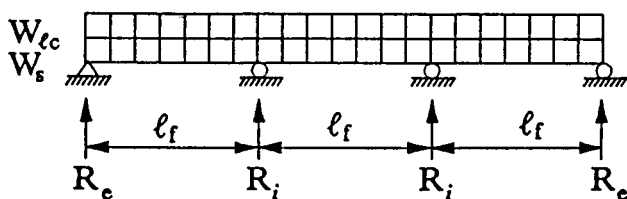
Double
span
condition



$$R_e = 0.375 \ell_f (W_{\ell_c} + W_s)$$

$$R_i = 1.25 \ell_f (W_{\ell_c} + W_s)$$

Triple
span
condition

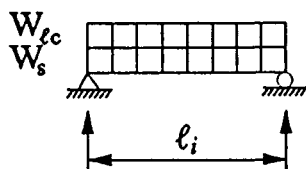


$$R_e = 0.4 \ell_f (W_{\ell_c} + W_s)$$

$$R_i = 1.25 \ell_f (W_{\ell_c} + W_s)$$

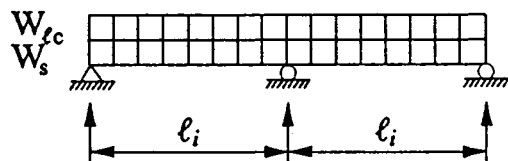
Figure 2.2 - Loading diagrams for support reactions ¹

Single
span
condition



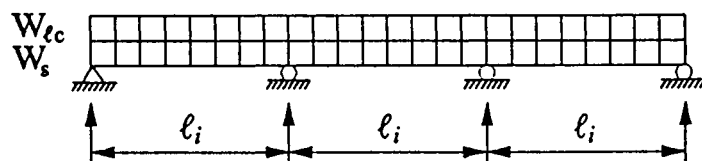
$$\Delta_1 = \frac{5W_s \ell_i^4}{384 E_s I_s}$$

Double
span
condition



$$\Delta_2 = 0.42 \Delta_1$$

Triple
span
condition



$$\Delta_3 = 0.53 \Delta_1$$

Figure 2.3 - Loading diagrams for deflections ¹

4 ¹ Unit width of slab in SI units shall be consistent with SI units for other terms.

2.3 - Composite section

2.3.1 - Strength design

2.3.1.1 - General. Full design loads shall be imposed upon the structure only after the concrete has reached full design strength. The allowable superimposed uniform live load shall be the smallest value obtained from all the design considerations presented in these criteria.

2.3.1.2 - Load factors.

(a) The computed uniform dead load, W_d , and the specified superimposed uniform live load, W_l , shall be multiplied by load factors to obtain the required factored uniform load, W_u , which shall be the greater of the following:

$$W_u = 1.4 W_d \quad (2-1)$$

$$W_u = 1.2 W_d + 1.6 W_l + 0.5 W_{rf} \quad (2-2)$$

where W_{rf} (if applicable) are the roof live loads (see also 4.11 of ASCE 7 [3], snow loads, or rain loads, except ponding).

(b) If resistance to wind loads, W , or to earthquake loads (perpendicular to the slab), E , is included in the design, W_u shall be:

$$W_u = 1.2 W_d + (0.5 W_l \text{ or } 0.8 W) + 1.6 W_{rf} \quad (2-3)$$

$$W_u = 1.2 W_d + 0.5 W_l + 1.3 W + 0.5 W_{rf} \quad (2-4)$$

$$W_u = 1.2 W_d + 1.5 E - (0.5 W_l \text{ or } 0.2 W_{sw}) \quad (2-5)$$

$$W_u = 0.9 W_d - (1.3 W \text{ or } 1.5 E) \quad (2-6)$$

whichever governs, where W_{sw} are the snow loads. The designer shall select the appropriate combination and shall consider other combinations in accordance with ASCE 7 [3], as needed.

2.3.1.3 - Strength reduction factors, ϕ .

(a) Shear-bond	0.75
(b) Flexure (underreinforced)	0.85
(c) Flexure (underreinforced when $f_u/f_y \leq 1.08$)	0.65
(d) Flexure (overreinforced)	0.70
(e) Flexure (plain concrete)	0.60

2.3.1.4 - Continuity over supports. In continuous slabs, those sections subjected to negative bending shall be designed as conventionally reinforced concrete slabs. In composite slabs, moments and shears shall be calculated by an analysis or, if applicable, by the coefficients of Chapter 8 of **ACI Building Code Requirements for Reinforced Concrete, ACI 318 [2]**. Those one-way slab segments that undergo positive bending shall be designed for shear-bond and flexure, and the strength of these segments shall be calculated accordingly. For interior spans, length of the segment of slab subject to positive moment, ℓ''_i , shall be substituted for span length of slab, ℓ_i , in the equations of Section 2.3.1.5.1 dealing with shear-bond failure. For shear-bond analysis of end spans, full span length, ℓ_i , shall be used.

2.3.1.5 - Strength relationships. The strength of a composite steel deck slab shall be governed by one of the following capacities:

- (a) Shear-bond,
 - (b) Flexure of an underreinforced section,
- or
- (c) Flexure of an overreinforced section.

2.3.1.5.1 - Shear-bond strength. The factored shear force, V_u , and the design shear-bond strength, ϕV_n , (in the direction parallel to deck corrugations) in lbs. per ft. of width of slab¹ are related as follows:

$$V_u \leq \phi V_n \quad (2-7)$$

For simply supported spans with concentrated loading:

$$\phi V_n = \phi \left[bd \left(\frac{m p d}{\ell_i} + k \sqrt{f'_c} \right) + \frac{\gamma W_s \ell_i}{2} \right] \quad (2-8)$$

For a uniformly applied load, Eq. (2-8) becomes:

$$\phi V_n = \phi \left[bd \left(\frac{4 m p d}{\ell_i} + k \sqrt{f'_c} \right) + \frac{\gamma W_s \ell_i}{2} \right] \quad (2-9)$$

The shear-bond tests from which the m and k values are derived shall properly account for the variables as indicated in Section 3.2.3.1.

¹ Unit width of slab in SI units shall be consistent with SI units for other terms.

Proper consideration shall be given to conditions of support during concrete placement to determine the portion of dead load carried by the composite section, as shown in Table 2.1.

Table 2.1 - Shoring factors

Support during concrete placement	γ^*
Complete support throughout length	1.0
Unshored	0.0
Shored at center	0.625
Shored at third points	0.733

* Other γ factors are needed for support cases not shown.

Based on Eq. (2-2) of Section 2.3.1.2, the permissible superimposed, uniform live load for shear bond, W_{ts} , in psf is determined from:

$$W_{ts} = \frac{1}{1.6} \left[\frac{2\phi V_n}{\ell_f} - 1.2 (\gamma W_s + W_{ds}) \right] \quad (2-10)$$

where ϕV_n is determined by either Eq. (2-8) or Eq. (2-9), and W_{tr} is zero.

For cases involving loadings formed from Eqs. (2-1), (2-3) through (2-6), W_{tr} , or concentrated loads, Eq. (2-10) shall be redefined.

2.3.1.5.2 - Flexural strength.

Composite slabs subject to flexural failure are classified as underreinforced or overreinforced slabs depending on reinforcement ratio $\rho = A_s/bd$. The ratio that denotes a balanced condition is:

$$\rho_b = \frac{0.85\beta_1 f'_c}{f_y} \left[\frac{\epsilon_c E_s (h - d_d)}{(\epsilon_c E_s + f_y) d} \right] \quad (2-11)$$

Slabs with $\rho \leq \rho_b$ are underreinforced, whereas slabs with $\rho > \rho_b$ are considered overreinforced.

Design of cross sections subject to flexure shall be based on:

$$M_u \leq \phi M_n \quad (2-12)$$

where M_u is factored moment at section considered and ϕM_n is design moment strength.

(a) Underreinforced case. The design moment strength, ϕM_n , in this case is:

$$\begin{aligned} \phi M_n &= \frac{\phi A_s f_y}{12} \left[d - \frac{a}{2} \right] \\ (\phi M_n &= \phi A_s f_y \left[d - \frac{a}{2} \right] \\ &\text{for SI system}) \end{aligned} \quad (2-13)$$

Eq. (2-13) shall be used only for slab cross sections capable of developing the yield stress across the entire deck depth. When the strain compatibility of the slab cross section or the ductility of the steel does not permit a yielding across the entire steel deck section, a general strength analysis shall be performed. Eq. (2-13) is not valid for decks consisting of a steel grade where f_u/f_y is less than 1.08. Further, Eq. (2-13) shall not apply when there is additional steel reinforcement above the steel deck, and when a partial deck section is subject to compression. For such cases where Eq. (2-13) does not apply, a general strain analysis shall be performed.

(b) Overreinforced case. The design moment strength, ϕM_n , in this case shall be found by a general strain analysis. Tests shall be conducted to validate the computed capacity, see Section 3.2.4.3.

Based on Eq. (2-2) of Section 2.3.1.2, the permissible superimposed uniform live load, W_{tr} , for flexure in psf is determined from:

$$W_{tr} = \frac{1}{1.6} \left(\frac{\phi M_n}{C_m \ell_f^2} - 1.2 (W_s + W_{ds}) \right) \quad (2-14)$$

where W_{tr} is zero.

For cases involving loadings formed from Eqs. (2-1) or (2-3) through (2-6), or W_{tr} , Eq. (2-14) shall be redefined.

2.3.2 - Service load design

2.3.2.1 - Section properties for deflection calculations. Composite flexural section properties needed to determine vertical deflections of composite slabs shall be computed in accordance with conventional elastic theory used in reinforced

concrete, requiring a transformation of steel areas to equivalent areas of concrete. The following assumptions shall be used in the derivation of the necessary relationships:

(a) Plane sections before bending remain plane after bending; i.e., the longitudinal strains of concrete and steel at any section transverse to deck corrugations are proportional to the distance of fibers from composite neutral axis.

(b) Stresses are proportional to strain in both concrete and steel at service loads.

(c) The entire steel cross section shall be utilized except as reduced by holes.

(d) Moment of inertia, I_d , used in deflection calculations shall be taken as the average of those of the cracked, I_c , and uncracked sections, I_u , using design depth of slab. Formulas for flexural section properties and moments of inertia are given in Appendix B.

2.3.2.2 - Deflection limitations.

Consideration shall be given to both immediate and long-term loading. Deflection provisions apply to composite slabs of all categories, and all calculated maximum deflections shall be based on the assumptions of Section 2.3.2.1. Maximum allowable computed deflections are listed in Table 2.2. The additional deflection caused by creep shall be calculated by multiplying the immediate deflection due to the sustained load by the factor:

$$\lambda = [2 - 1.2(A'_s/A''_s)] \geq 0.6 \quad (2-15)$$

2.3.3 - Special design considerations

2.3.3.1 - Control of shrinkage and temperature effects. Minimum control of shrinkage and temperature effects shall be in accordance with the following:

Transverse reinforcement bars having a yield strength of at least 60,000 psi (400 MPa) or welded wire fabric shall be provided. Area of this reinforcement shall be equal to 0.00075 times the area of concrete above the steel deck but not less than 0.028 sq. in. per ft. of width (60 mm² per m of width). Wires welded directly to top of the deck (transverse to corrugations) for the purpose of providing shear transfer shall be considered for shrinkage reinforcement.

2.3.3.2 - Punching shear. Punching shear due to concentrated point loads shall be considered. The general procedure given in Section 11.12.2 of the ACI Building Code Requirements for Reinforced Concrete, ACI 318 [2] shall be used, with the following modification. The depth of concrete over top corrugation of deck, h_c , shall be considered as effective depth for punching shear calculations, and nominal shear stress carried by the concrete shall not exceed $2 \sqrt{f'_c}$ ($(1/\phi) \sqrt{f'_c}$ MPa). Alternately, performance tests shall be used to determine limiting punching shear, provided the test method and allowable shear values are established by the Engineer and approved by the authority having jurisdiction.

2.3.3.3 - Two-way action. In composite slabs requiring two-way action for load distribution, the flexural strength in the direction transverse to deck corrugations shall be considered based on

$$M_u \leq \phi M_n \quad (2-16)$$

where M_u is factored moment at section considered and ϕM_n is design moment strength. Two cases apply for determination of design moment strength ϕM_n :

(a) When no supplementary transverse reinforcement is provided, flexural strength shall be taken as that of plain concrete section above the deck corrugations. Any contribution from the steel deck shall be neglected. The nominal moment strength, M_n , in foot-pounds per foot of width is:

$$M_n = \frac{f_r h_c^2}{6} \quad (2-17)$$

where $\phi = 0.60$ (see 2.3.1.3)

(b) When supplementary transverse reinforcement is provided in the tension zone, Section 2.3.1.5.2 shall be used. The area of steel, A_s , shall consist entirely of the supplementary reinforcing, and only the section above the deck corrugations shall be considered effective unless tests indicate that other configurations are acceptable.

Table 2.2 - Maximum allowable deflections under service loads [2]

Type of composite slab	Type of deflection	Deflection limitation (inches)
Flat roofs not supporting or attached to nonstructural elements likely to be damaged by large deflections	Immediate deflection due to the live load W_{lf}	$\frac{\ell_i}{180}$ ¹
Floors not supporting or attached to nonstructural elements likely to be damaged by large deflections	Immediate deflection due to the live load W_l	$\frac{\ell_i}{360}$
Roof or floor slabs supporting or attached to nonstructural elements likely to be damaged by large deflections	That part of the total deflection which occurs after attachment of the nonstructural elements, the sum of the long-time deflection due to immediate deflection due to	$\frac{\ell_i}{480}$
Roof or floor slab to nonstructural elements not likely to be damaged by large deflections	any additional live load ²	$\frac{\ell_i}{240}$ ³

¹ Limit not intended to safeguard against ponding. Ponding shall be checked by suitable calculations of deflection, including added deflection due to ponded water, and by considering long-time effects of all sustained loads, camber, construction tolerances, and reliability of provisions for drainage.

² The long-time deflection shall be reduced by the amount of deflection which occurs before attachment of the nonstructural elements. This amount shall be determined on the basis of accepted engineering data related to the time-deflection characteristics of members similar to those being considered.

³ This limit cannot be greater than the tolerance provided for the nonstructural elements if based on accepted engineering data.

2.3.3.4 - **Repeated or vibratory loading.** Where repeated or vibratory loading is a factor, adequate test data shall be obtained to substantiate the suitability of the steel deck slab involved.

CHAPTER 3 - PERFORMANCE TESTS

3.1 - Introduction

Data from performance tests shall be used to determine strength and behavior of composite slabs in order to establish allowable design loads. The tests shall include full-scale slab elements and strength determinations of the component materials.

3.2 - Testing of composite slab elements

3.2.1 - Specimen preparation

3.2.1.1 - **General.** The test model assembly and load locations are such that shear-bond failures are achieved on short spans and flexural failures are achieved on long spans. During construction of test specimens, resulting stresses and deflections shall not exceed values indicated in Sections 2.2.4 and 2.2.6. The coating of all steel deck units shall be in a condition simulating as closely as possible that found in actual field practice. Concrete used in the composite specimens shall be prepared and cured in accordance with standard construction requirements stated in applicable sections of the **ACI Building Code Requirements for Reinforced Concrete, ACI 318 [2]**.

3.2.1.2 - **Dimensions of composite specimens.** Dimensions of test specimens within a particular group shall be determined as follows:

(a) **Length.** Test specimen lengths shall include the entire range of spans found in field applications of the tested deck. In lieu of utilizing span lengths, consideration shall be given to testing shear spans in accordance with Table 3.1.

(b) **Width.** Width b_d of all slab specimens shall be at least equal to one steel deck panel but not less than 2 ft. (600 mm).

(c) **Depth.** Slab depths shall encompass the range found in field applications of the tested deck. Appendix C shall be used in determining the actual dimensions.

3.2.2 - **Test procedure.** Two primary parameters to be determined from tests are:

- (a) Maximum load capacity, and
- (b) Mode and details of failure.

3.2.2.1 - **Loading of specimens.** All specimens shall be tested as shown in Figure 3.1. Alternatively, if uniform loading is used in lieu of concentrated loads, ℓ'_i shall be taken as the distance from the reactive support to the observed major failure crack. Load shall be applied in maximum increments of one-tenth of the estimated failure load. Load shall be applied continuously and without shock, except for pauses to take instrumentation readings at the prescribed load increments. Rate of loading shall not increase extreme compressive fiber stress in the concrete by more than 150 psi/min. (1 MPa/minute).

Table 3.1 - Limiting values of depths and shear spans

Region*	Depth, h	Shear span, ℓ'_i
A	Minimum practical depth $\geq 3\text{-}1/2$ in. (90 mm)	> 36 in. (900 mm), but $P_e \ell'_i / 2 < 0.9 M_n$, determined from Section 2.3.1.5.2 with $\phi = 1.0$
B	Maximum practical depth	≥ 18 in. (450 mm), and $\leq$ test section width

* See Sections 3.2.3.1 and 3.2.4.2 and Figure 3.2.

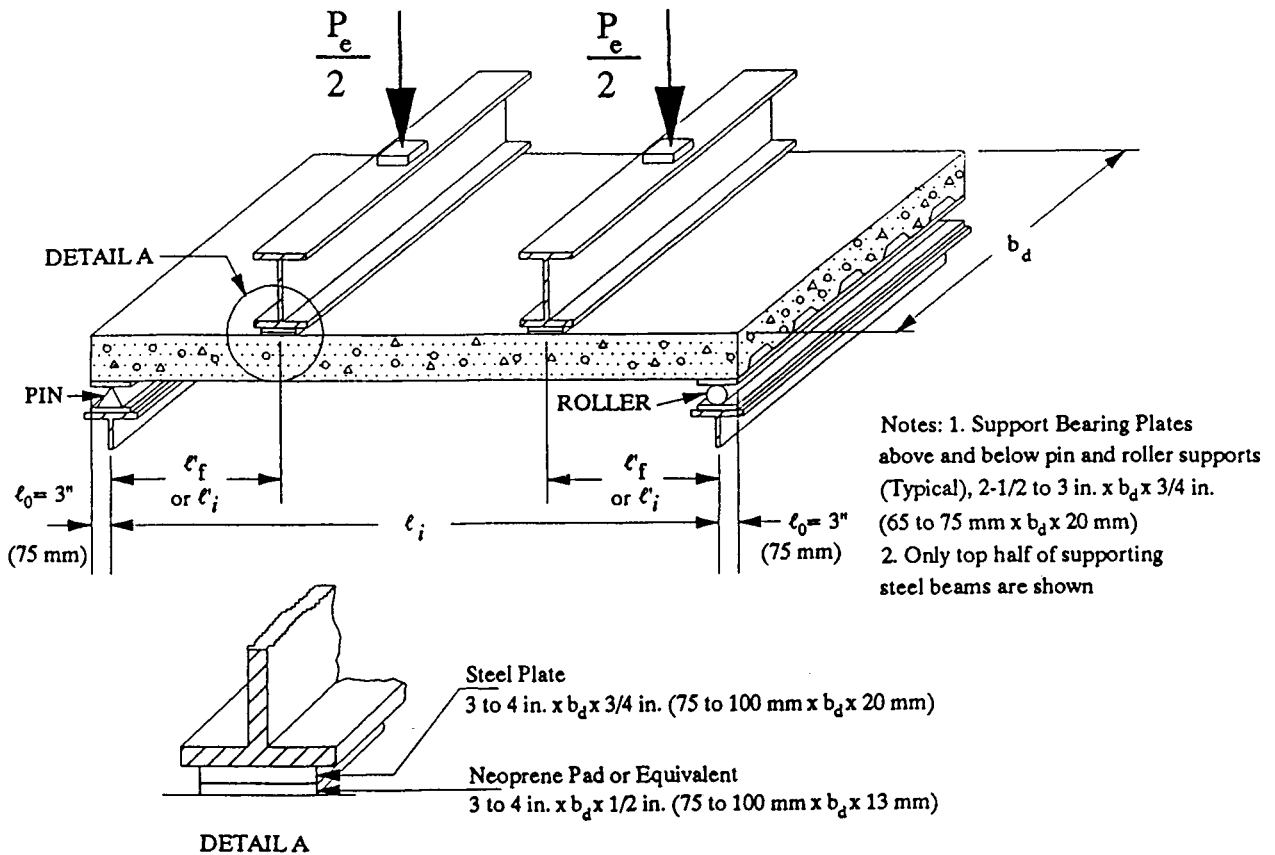


Figure 3.1 - Typical test assembly

3.2.2.2 - Instrumentation. Load-measuring equipment shall be calibrated within ± 1 percent. Midspan deflections and end slip between steel deck and concrete shall be recorded to the nearest 0.001 in. (0.01 mm) at each increment of load.

3.2.2.3 - Recording of data. The following data shall be recorded and documented for each test specimen:

(a) **Dimensional properties:** (values in S.I. units are also acceptable)

b_d = width of composite test slab, ft.¹
 h = out-to-out depth of slab measured at least at ends, center, and at 1/4 points or load points, in; measurements shall be taken on an interior rib and at edges of specimen.

d_d = overall depth of steel deck profile, in.
 ℓ_f = length of span or shored span, ft.
 ℓ'_i = length of shear span, in.
 s = center-to-center spacing of shear transfer devices, in.
 t = thickness of steel deck exclusive of coating determined from specimens used to obtain material properties, in; for cellular decks, each sheet shall be measured.
 h_i = out-to-out depth of slab at failure crack, in.
 ℓ_o = slab overhang at supports, in.

Dimensional characteristics of shear devices, such as for embossments, the embossment length, width, depth, spacing, shape configuration, and general variation in depth and size shall be recorded - See Appendix C.

¹⁰ ¹ Unit width of slab in SI units shall be consistent with SI units for other terms.

(b) Material properties:

- f'_c = compressive test cylinder strength of concrete at time of slab testing, psi, according to ASTM C39 [4]
 f_y = measured yield strength of steel, psi, ASTM A370 [4]
 f_u = measured tensile strength of steel, psi, ASTM A370 [4]

The percent elongation of steel coupon by ASTM A370 [4], the steel specification and grade and other material properties shall be recorded.

(c) Unit loads:

- W_{dd} = steel deck dead load, psf
 W_{dc} = concrete dead load including additional weight of concrete added due to deck deflection, psf

(d) Construction conditions:

- (1) steel surface coating and condition - if galvanized, state ASTM type of coating used and whether chromate or other conversion coating was used, and if so what type; if painted, state type of paint; state any surface deterioration or weathering of the coating;
- (2) shoring;
- (3) concrete mix design and date of casting;
- (4) type and location of steel welded wire fabric; and
- (5) concrete cylinder air dry density at time of testing.

(e) Test data: A brief description of significant events during testing, including test date, shall be recorded along with an identification of the failure mode and details of failure. The load P_u , the engineer, and the technician responsible for testing shall be recorded.

In addition, the following shall be recorded:

- (1) midspan load-deflection measurements, Δ ,
- (2) load and end-slip measurements,
- (3) load at first observable crack, and
- (4) for cellular sections - thickness of each component element, number of rows, spacing, and type of fastening between components.

3.2.3 - Scope of tests

3.2.3.1 - Shear-bond tests. To develop m and k values of Section 2.3.1.5.1, the following procedures shall be used:

(a) Number of slab specimens shall be adequate to cover range of variables involved and to provide sufficient data for design in accordance with these criteria.

(b) Tests shall be conducted on each deck profile for which design values are needed. Tests shall be conducted on each design thickness for each profile or on a combination of thicknesses as indicated in Section 3.2.4.2.

(c) For cellular decks, the thickness of the bonded formed section shall be used in Section 3.2.4.2, applying the limit indicated in (b) above. In lieu of performing the test series, the A_v value used in determining the superimposed load capacity for a system using only cellular deck sections with shear-bond governing shall include the bottom element of cellular decks provided that the results from a minimum of two tests indicate that a shear-bond strength lower than the predicted does not exist. In the event that the confirmatory tests are lower than the predicted, then the entire test series shall be performed. The designer shall choose to perform the entire test series for m and k values.

(d) Tests shall be made to determine the effects of all surface coatings or finishes to be used on a given deck profile. In lieu of testing all surface finishes, the shear-bond strength shall be obtained from tests of a coating demonstrating a lower strength, provided that verification of a coating's lower shear-bond strength is found from a minimum of two comparison tests.

(e) Tests using specimens made with lightweight structural concrete shall be used to establish values for other types of concrete having equal or greater densities, provided that at least two comparison tests indicate that shear-bond strength is lower for those slabs containing lightweight concrete.

(f) Tests shall be made on at least two specimens in each of two regions, A and B as defined in Table 3.1, to develop shear-bond data. Region A defines specimens with small slab depths, h , and/or relatively large shear spans, ℓ'_1 , whereas Region B defines specimens with large depths and/or relatively small shear spans.

(g) When tests in either of the two regions of Table 3.1 consist of only two composite slab specimens, see Section 3.2.4.2(b).

(h) For decks on which spacing of shear transfer devices, s , varies from one deck section to another, a series of tests to determine m and k is required for each spacing.

(i) Tests are not required for all concrete compressive strengths. For those cases where the usage strength of concrete is lower than that tested, the shear-bond strength shall be prorated according to:

$$\frac{V_{c1}}{V_{c2}} = \left(\frac{f'_{c1}}{f'_{c2}} \right)^{1/4} \quad (3-1)$$

For those cases where usage strength of concrete is higher than that tested, the shear-bond prediction in Section 2.3.1.5.1 shall be used to arrive at an adjusted capacity for changes of f'_c not exceeding 1500 psi (10 MPa). Strength of concrete for those specimens tested shall not be less than 2500 psi (17 MPa).

3.2.3.2 - Flexural tests. For decks deeper than a nominal 3 in. (75 mm) or formed from steels with low ductility having a ratio f_u/f_y less than 1.08, and a total elongation of less than 10 percent for a 2-in. (50 mm) gage length or less than 7 percent for an 8-in. (200 mm) gage length tested in accordance with ASTM A370 [4], a minimum of three representative tests shall be conducted to establish the applicability of the design moment (Eq. (2-13) or the general strain analysis) for the deck in question, (see Section 3.2.4.3). For those cases where Eq. (2-13) does not apply, a minimum of three representative tests shall be performed. The specimens tested shall cover the minimum and maximum reinforcement ratios used in actual construction.

3.2.4 - Test result evaluation

3.2.4.1 - General. Evaluation of results from tests developed in Section 3.2.3 shall be performed to establish design expressions required for application of provisions of Chapter 2.

3.2.4.2 - Shear-bond. To evaluate the test results for shear-bond strength, the following procedure shall be used:

(a) A plot of $V_e/bd\sqrt{f'_c}$ versus $\rho d/\ell_i\sqrt{f'_c}$ as shown in Figure 3.2 shall be developed for each representative steel deck profile category outlined in Section 3.2.3.1. A shear-bond line, using the

required test data, is then established for each plot to determine slope, m_1 , and ordinate intercept, k_1 , of the line. Computation for d is based on the depth at the failure crack, h_f .

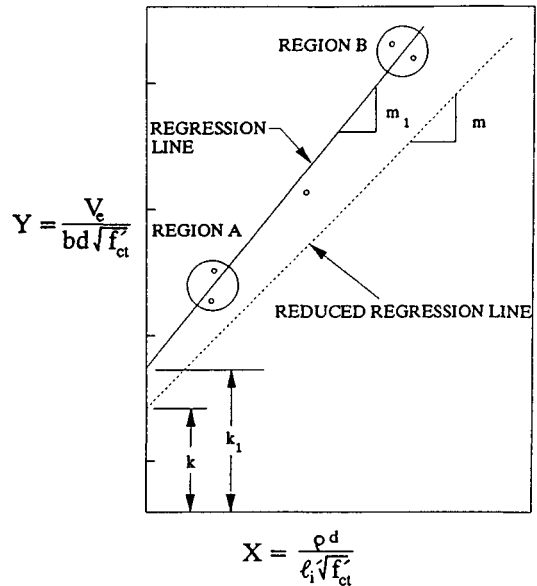


Figure 3.2 - Typical shear-bond plot showing the reduced regression line for m and k

(b) If only two tests have been performed in either the A or B region as defined in Table 3.1, and

the value of $V_e/bd\sqrt{f'_c}$ for either test deviates from the average of the two by more than ± 15 percent, at least two additional tests of the same kind shall be performed and the two lowest results shall be used in determining the shear-bond line, unless an inadequate specimen fabrication or test procedure dictates otherwise.

(c) Values m_1 and k_1 obtained from Figure 3.2 shall be reduced by 15 percent as indicated by the reduced shear-bond regression line in Figure 3.2 to obtain m and k , respectively, for use in design expressions given in Chapter 2. If eight or more tests are used in the analysis; values of m_1 and k_1 shall be reduced by only 10 percent to obtain m and k , respectively.

(d) When test results for more than two values of $\rho d/\ell_i\sqrt{f'_c}$ are plotted, a regression

analysis shall be used to determine the shear-bond line.

(e) In lieu of testing all deck thicknesses, shear-bond strength obtained from tests on a thinner

deck shall be used for any thicker deck provided that results from a minimum of two tests on thicker decks with the same profile as the thinner deck indicate that lower shear-bond strength does not exist. In addition, if tests on a thicker deck are conducted using the prescribed procedure in Figure 3.2, shear-bond strength for any intermediate deck thickness shall be obtained by linear interpolation. Extrapolation is not permitted; however, the use of m and k values established for any steel deck design thickness category shall be limited to steel thicknesses up to 0.003 in. (0.076 mm) smaller than the mean thickness tested. Anticipated thickness of steel shall be used in the design shear-bond prediction in Section 2.3.1.5.1.

(f) Alternate shear-bond analysis. Other rational methods for establishing the shear-bond strength shall be used if the pertinent parameters contributing to a shear-bond failure (including deck cross section; steel thickness; shear span; concrete weight, strength, and type; shear transfer devices; and method of loading) are considered. These methods include nonlinear relationships between the various parameters. Sufficient tests shall be made to establish the method-test variability. One such alternate shear-bond approach is given in Appendix D.

3.2.4.3 - Flexure. Where flexure tests are required by Section 3.2.3.2, the evaluation of test results of flexural failure mode shall be based on a comparison of experimental and calculated strengths. For those cases where Eq. (2-13) does not apply, tests required in Section 3.2.3.2 shall be analyzed by a general stress and strain analysis. If the test moment strength is less than 95 percent of the moment strength calculated in accordance with Section 2.3.1.5.2, at least three additional tests shall be performed. For purposes of this comparison, ϕ

shall be taken equal to 1.0. Flexural failure mode shall be verified by inspections of the tested sample and end-slip, deflection, and strain data.

3.2.4.4 - Design dimensions of the steel deck. The design arrangement and depths of shear transfer devices shall be established from tests by the manufacturer.

3.3 - Existing tests

Data from tests predating these criteria shall be used when evaluated in conformance with Section 3.2.4. Such previous test data are not required to be in Regions A and B as defined in Table 3.1 but must be approved by a registered professional engineer. Predated data shall only be used to fulfill the requirements for number of tests, when at least one test falls in Region A and one in Region B.

3.4 - Performance tests

3.4.1 - General. When required to verify the structural performance of a composite steel deck floor system through in-situ load tests, the engineer shall establish existing dead loads as well as anticipated dead and live loads, and the deflection limits.

3.4.2. - Acceptance test. The acceptance test shall include all dead loads and live loads, multiplied by the appropriate factors from Section 2.3.1.2, and for acceptance the applied test load shall not be less than 85 % of the factored loads. The applied test load shall be maintained for not less than 24 hours and the measured deflections shall be within limits set by the engineer.

APPLICABLE DOCUMENTS

1. *Specification for the Design of Cold-Formed Steel Structural Members*, American Iron and Steel Institute, 1101-17th Street N.W., Washington, D.C., 20036, August 19, 1986, with December 11, 1989 addendum.
2. *ACI Standards*, American Concrete Institute, P.O. Box 19150, Redford Station, Detroit, Michigan, 48219.

ACI 318-89, *Building Code Requirements for Reinforced Concrete*, 1989
or the Metric Version
ACI 318M-89, *Building Code Requirements for Reinforced Concrete (Metric-SI)*, 1989
3. *ASCE 7-88, Minimum Design Loads for Buildings and Other Structures*, American Society of Civil Engineers, 345 E. 47th Street, New York, NY 10017 (formerly ANSI A58.1-1982, American National Standards Institute, Inc., New York, New York, 10017-2398.), 1990.

4. *ASTM Standards*, American Society for Testing and Materials, 1916 Race Street, Philadelphia, Pennsylvania, 19103.

(Note: Some of the ASTM standards may have SI (metric) unit editions available. If the Standard is used to design in SI units, the user shall investigate whether an SI unit edition of an ASTM standard is available.)

ASTM A370-77, Standard Methods and Definitions for Mechanical Testing of Steel Products (Editorial changes 1979 and 1980)
ASTM C39-86, Standard Test Methods for Compressive Strength of Cylindrical Concrete Specimens, 1986

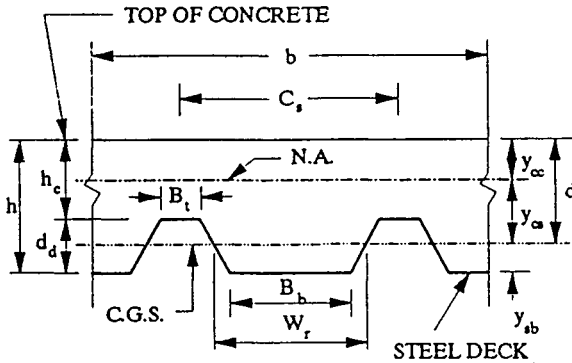
APPENDIX A - SI UNITS

Table A.1 - SI Conversion factors

	TO CHANGE	MULTIPLY BY
LENGTH	in to mm ft to mm ft to m	25.4 (exact) 304.8 (exact) 0.3048 (exact)
AREA	in ² to mm ² ft ² to m ²	645.16 (exact) 0.092903
MASS	lb to kg 2000 lb to 1000 kg lb/ft to kg/m lb/ft ³ to kg/m ³ lb/yd ³ to kg/m ³	0.453592 0.907185 1.48816 16.0185 0.593276
FORCE	lb to N kip to kN lb/in to N/m lb/ft to N/m kip/ft to kN/m psf to kN/m ²	4.44822 4.44822 175.127 14.5939 14.5939 47.880
PRESSURE	lb/in ² to kPa lb/ft ² to kPa kip/in ² to MPa	6.89476 0.04788 6.89476
SECTION MODULUS	in ³ to mm ³ in ³ /ft to mm ³ /m	16387.1 53763.5
MOMENT OF INERTIA	in ⁴ to mm ⁴ in ⁴ /ft to mm ⁴ /m	416231 1365587

APPENDIX B - SECTION PROPERTIES FOR CALCULATING DEFLECTIONS OF COMPOSITE SLABS

B.1. - Transformed composite neutral axis. The distance y_{cc} from the extreme compression fiber of the concrete to the neutral axis of the transformed composite section shall be determined from Figure B.1.¹



- Note:
1. Substitute h_c for h when evaluating test results.
 2. Section shows non-cellular deck. Section shall be either cellular, a blend of cellular and non-cellular, or non-cellular deck.
 3. C.G.S. = centroidal axis of full cross section of steel deck
 4. N.A. = neutral axis of transformed composite section

Figure B.1 - Composite Section

B.2 - Moment of inertia of cracked section. When y_{cc} is equal to or less than the depth of concrete, h_c , above the top of steel deck, that is, $y_{cc} \leq h_c$, then

$$y_{cc} = d \{ [2\rho n + (\rho n)^2]^{1/2} - \rho n \} \quad (B-1)$$

If $y_{cc} > h_c$, use $y_{cc} = h_c$

The cracked moment of inertia I_c is:

$$I_c = \frac{b}{3} (y_{cc})^3 + nA_s (y_{cs})^2 + nI_{s'} \quad (B-2)$$

B.3 - Moment of inertia of uncracked section. For the uncracked moment of inertia:

$$y_{cc} = \frac{0.5bh_c^2 + nA_s d + W_r d_d (h - 0.5d_d) b / C_s}{bh_c + nA_s + W_r d_d b / C_s} \quad (B-3)$$

The uncracked moment of inertia is:

$$I_u = \frac{bh_c^3}{12} + bh_c (y_{cc} - 0.5h_c)^2 + nI_{s'} + nA_s y_{cs}^2 + \frac{W_r b d_d}{C_s} \left[\frac{d_d^2}{12} + (h - y_{cc} - 0.5d_d)^2 \right] \quad (B-4)$$

B.4 - Moment of inertia of composite section. The moment of inertia of composite section considered effective for deflection computations is given by

$$I_d = \frac{I_u + I_c}{2} \quad (B-5)$$

APPENDIX C - DECK MEASUREMENTS

C.0 - Notation

- B_c = repeating width of one cell, in.
- d_d = overall depth of steel deck profile, in.
- D_w = length of web segment of profile cell, in.
- N_b = length of embossment along its base, in.
- N_t = length of embossment along its top ridge, in.
- $n_1, n_2, \text{etc.}$ = dimension along each straight segment of curved embossments, in.
- p_h = embossment depth, in.
- q = horizontal distance between embossment centerline, in.
- s = length of repeating embossment pattern, in.
- w = width of embossment, in.
- W_b = width of profile cell across its base, in.
- W_t = width of profile cell across its top, in.

C.1 - General. Predicting the superimposed slab capacity based on shear-bond using the equations in this Standard requires the measurement of the dimensions of the embossment and its orientation on the deck sheet.

C.2 - Measurements

C.2.1 - Embossments. Measurement of embossments shall be taken equally on all rows of embossments on all elements having embossments. A minimum of two embossments per row per element per test specimen shall be measured. Corresponding measurements of different rows shall be averaged.

C.2.2 - Measuring devices. Measurements shall be taken using either a dial gage or a depth micrometer with a tip tapered to 0.015 in. (0.381 mm) to accurately measure depth. A straight edge shall be used as a reference from the crest or ridge of adjacent embossments so that the measurement is taken on the raised embossment side of the sheet. Alternatively, measurement shall be taken directly from the recessed side of the embossment.

C.2.3 - Dimensions to be recorded for straight embossments. For straight embossments, dimensions to be recorded are the embossment depth, (P_b), length of embossment along the base (N_b) and along the ridge (N_r); width along the base (B_b) and width along the ridge (B_r); and the horizontal distance (s) between embossment centerlines. See Figure C.1.

C.2.4 - Dimensions to be recorded for curved embossments. For curved embossments, dimensions to be recorded are the distances along each straight segment of the base of the embossment (n_1 , n_2 , etc). These dimensions are added together to calculate N_b and N_r instead of attempting measurement. See Figure C.1.

APPENDIX D - AN ALTERNATE METHOD FOR CALCULATING FLEXURAL CAPACITY OF COMPOSITE SLABS.

D.1 - Introduction

This appendix has been developed in in.-lb. units¹. This appendix applies to composite deck slab

construction made from normal weight or lightweight structural concrete. The grades of steel used in fabricating the required confirmation tests shall be as described in Section A3 of the AISI Specification for the Design of Cold-Formed Steel Structural Members [1].

Composite slab bending strength is determined by the tension and compression forces developed in the cross section. Along the shear span, ℓ'_1 , bond force transfer limits the maximum tension forces that can be developed. The calculated bending strength model establishes a theoretical bending moment, M_{et} , at which steel yield first develops. M_{et} then is modified to $M_t = K M_{et}$ where K limits the tension force to the maximum that can be resisted by deck surface bond along the shear span.

D.2 - Calculated bending strength

D.2.1 - General. The bending moment tension forces per cell width, C_s , are calculated following Figure D.1. T_1 , T_2 , and T_3 are limiting steel forces set for the top element, the two webs, and the bottom surface respectively for a single cell unit C_s wide.

For each cell, the maximum reference bending moment at first yield, is

$$M_{et} = (T_1 e_1 + T_2 e_2 + T_3 e_3)/12. \quad (D-1)$$

$$e_3 = h - y_{\infty}/3 \quad (D-2)$$

$$e_2 = e_3 - d_d/2 \quad (D-3)$$

$$e_1 = e_3 - d_d \quad (D-4)$$

$$T_1 = f_y (B_t t) [(h - y_{\infty} - d_d)/(h - y_{\infty})] \quad (D-5)$$

$$T_2 = f_y (2D_w t) [(h - y_{\infty} - d_d/2)/(h - y_{\infty})] \quad (D-6)$$

$$T_3 = f_y (B_b t) \quad (D-7)$$

$$y_{\infty} = d \{ [2\rho n + (\rho n)^2]^{0.5} - \rho n \} \quad (D-8)$$

If $y_{\infty} \geq h_c$, use $y_{\infty} = h_c$

¹ Unit width of slab in SI units shall be consistent with SI units for other terms.

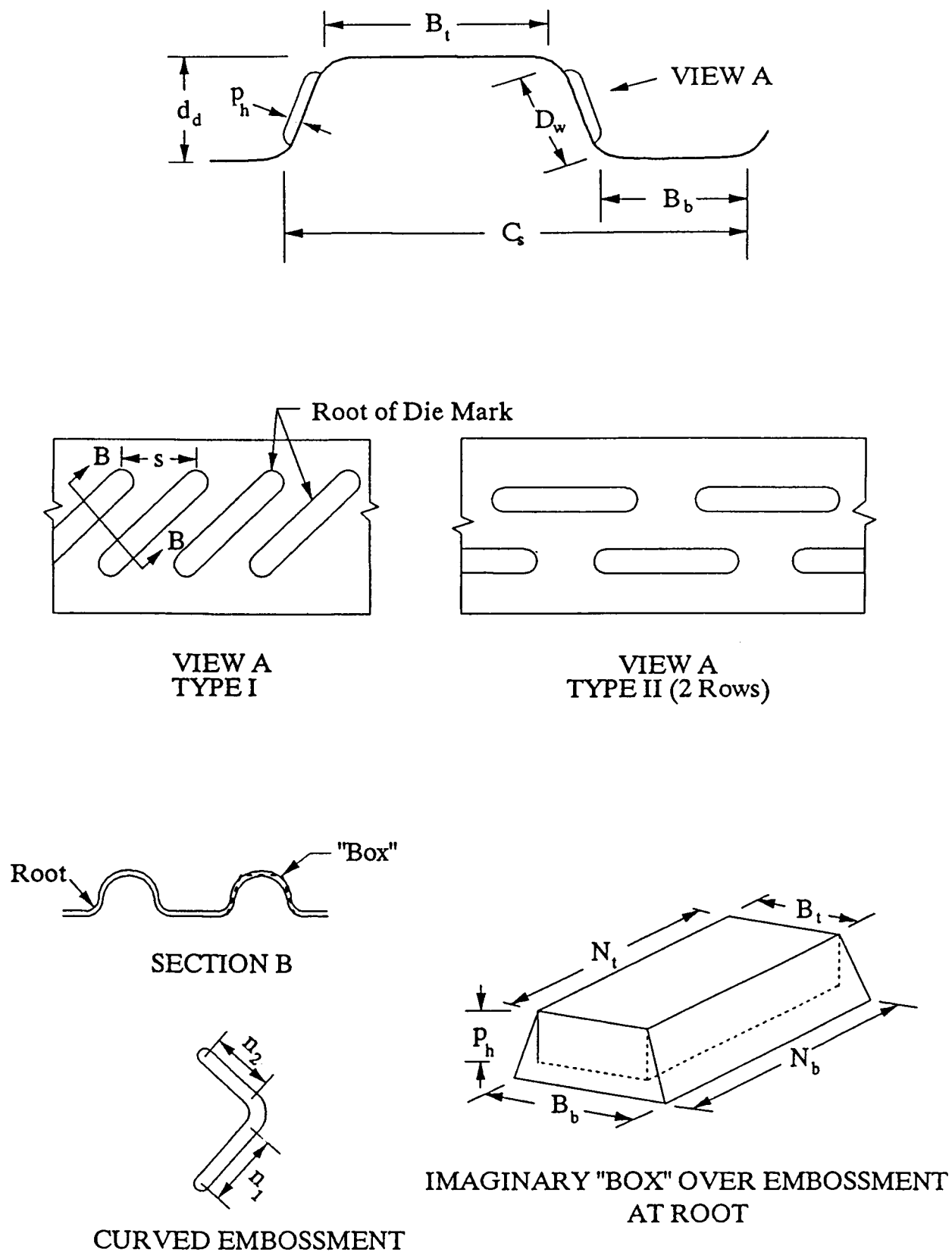


Figure C.1 - Deck measurements

D.2.2 - Shear Force Transfer. The shear-transferred anchorage forces, developed along the shear span, depend on the number of cell widths in the test slab of width b_d , on the steel section depth, and on the types of embossments. The calculated bending moment, M_i , is

$$M_i = K M_{ei}(12/C_s) \quad (D-8)$$

where $K = K_3/(K_1 + K_2)$ (D-9)

K_3 establishes the increase in efficiency, with increasing slab width, of average bond transfer per cell. With $N = 12 b_d/C_s$ being the number of cells in the test slab width:

$$K_3 = 0.87 + 0.0688(N) - 0.00222(N)^2 \leq 1.4 \quad (D-10)$$

K_1 measures the influence of the steel section depth on bond development along the shear span:

$$K_1 = [d_s/7.8]^{0.5} \quad (D-11)$$

K_2 is an indicator of mechanical bond performance along the shear span ℓ'_i and depends on the type of embossment patterns used.

D.2.2.1 - Type I Decks. Embossments for this type of deck shall be curved or straight and be oriented across the web width (See Fig. D.2A):

$$K_2 = \frac{D_w^{0.8} K_3 SS1}{1.0 + 60 (p_h^2 p_s^{1/3})} \quad (D-12)$$

$$SS1 = (3\ell_{nf}/70)(\ell_{nf}-14) + 3.6 \quad (D-13)$$

$$p_s = 12 \ell_e/s \quad (D-14)$$

D.2.2.2 - Type II Decks. Decks of this type have horizontal embossments oriented parallel to the deck span width (See Fig. D.2B)

$$K_2 = 627 t^2 SS2/(d_d e^x) + t(7/d_d)^2 \quad (D-15)$$

where e^x is an exponential function with $x = 25 p_h$. For Type II Decks, K_2 is dependent on the concrete strength, f'_c , and on the span to shear-span ratio:

$$SS2 = \frac{f'_c}{5000} + \frac{(12\ell_f/\ell'_i)^{0.5}}{3.6} \quad (D-16)$$

D.2.2.3 - Type III Decks. These decks with mixed embossment patterns, repeating at a spacing s , are treated as Type I decks but with a modified embossment intensity factor (See Fig. D.2C):

$$p_s = 12(N_v \ell_e + N_h w)/s \quad (D-17)$$

D.2.3 - Limitations. The formulas of Appendix D shall be limited to galvanized steel decks with Type I, II, or III embossment patterns. The embossment patterns shall be typical of the manufactured steel deck with the embossment height, p_h , between 0.035 in. (0.89 mm) and 0.105 in. (2.67 mm). The web angle, θ , shall be limited to values between 55° and 90° and the webs shall have no re-entrant bends in their flat width. The steel section depth, d_s , is limited to 3 in. (76 mm) or less and the cell spacing, C_s , to a maximum of 12 in. (305 mm). The test specimens shall have concrete with typical specified design compressive strengths limited between 2500 psi (17 MPa) and 6000 psi (40 MPa).

D.3 - Confirmation tests

The bending capacities, developed according to this appendix, shall be confirmed with at least three slab tests for a particular steel deck profile and given embossment pattern. At least two different steel deck thicknesses, t , shall be used with varying slab depths, h . The slabs shall be formed on two widths of steel deck units attached directly to a supporting structure with end fasteners of typical design and spacing.

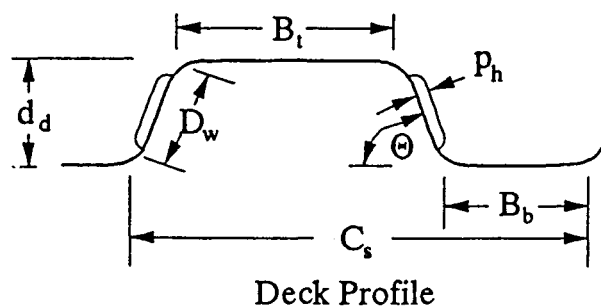
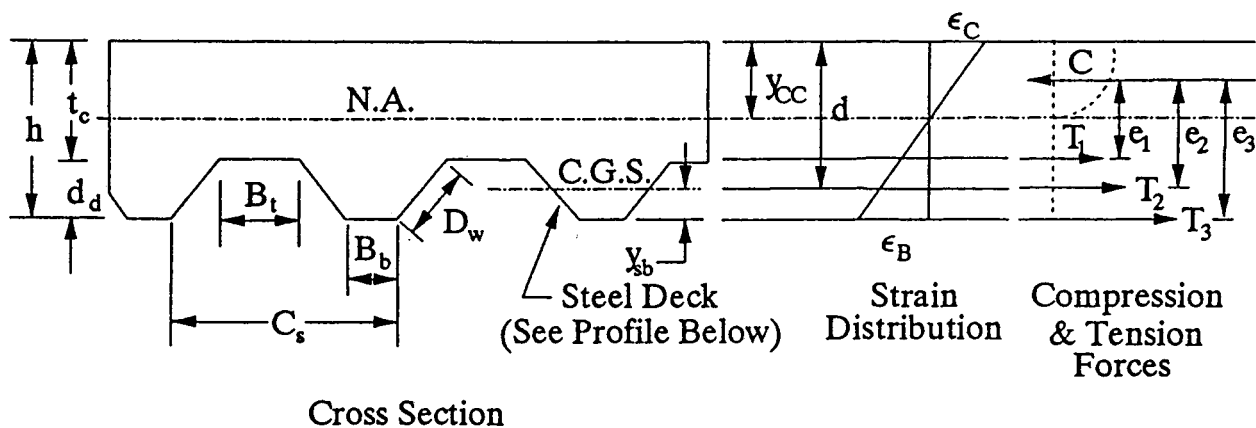
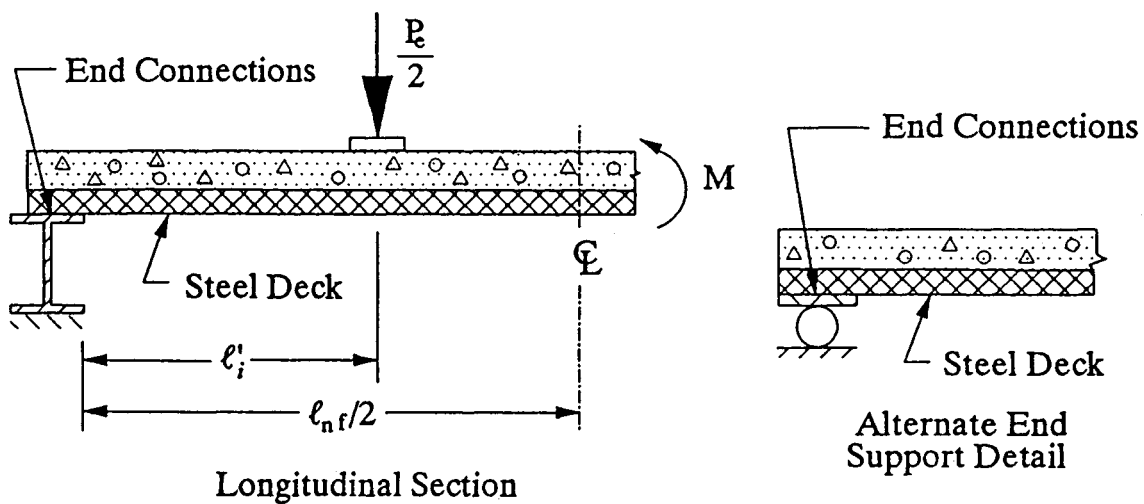
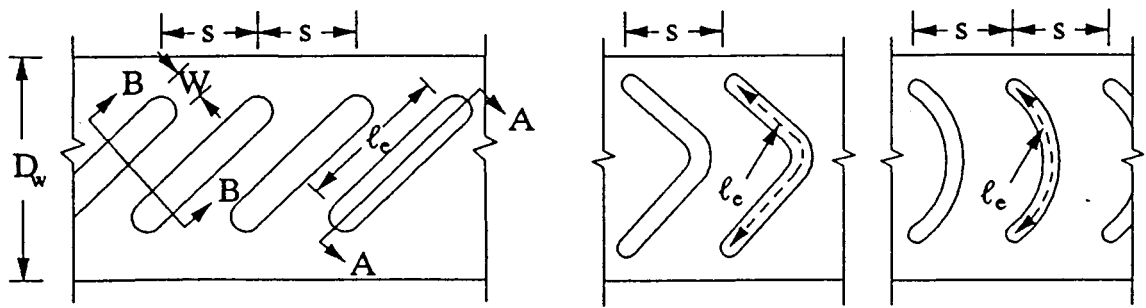
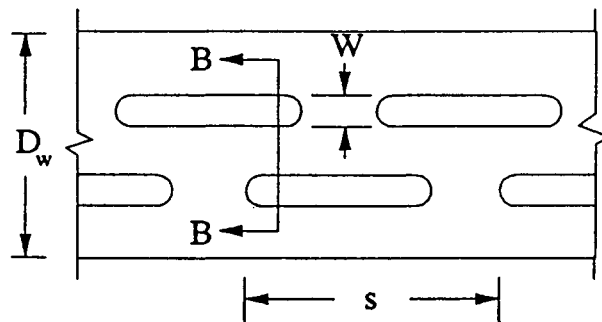


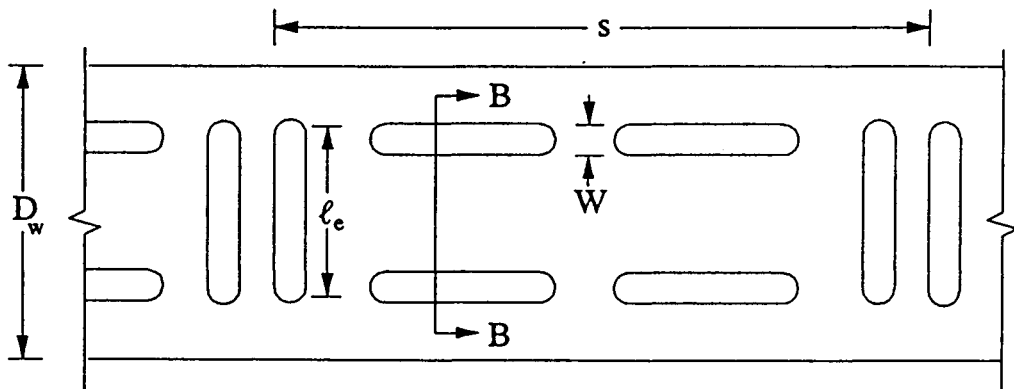
Figure D.1 - Deck sections



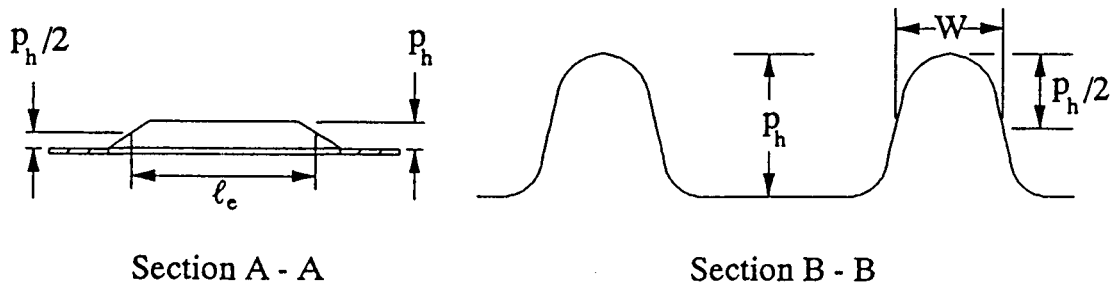
A. Type I Embossments with length measured along centerline



B. Type II Embossments



C. Type III Embossments



Section A - A

Section B - B

Figure D.2 - Embossments details

Testing shall be conducted with loading programs as described in Section 3.2.2 with line loads across the slabs at the span third-points to determine the bending moment strength, M_o , for each specimen:

$$M_o = (P_e/2)(\ell'_i/12)/b_d + M_s \quad (D-18)$$

M_s shall be established for each of the tested specimens following the procedures in this appendix. For each specimen, M_o/M_s shall be determined and, for the test series, the average M_o/M_s established as K_o . If the M_o/M_s result from each test is within $\pm 10\%$ of the average value K_o , the bending moment, M_o , shall be multiplied by K_o to establish the nominal moment strength M_n :

$$M_n = K_o M_o \quad (D-19)$$

If the M_o/M_s ranges are outside the $\pm 10\%$ limit, K_o shall be established in accordance with one of the following procedures: (a). A second series of three

tests shall be made with K_o being the average M_o/M_s values from four tests but excluding the two with the highest and lowest M_o/M_s values, or (b). The K_o factor shall be established from the two tests with the lower M_o/M_s values in the first series.

D.4 - Design equations

The nominal moment strength equation for M_n , as confirmed in Section D.3, shall represent the tested profile.

When $KK_o \geq 1.0$, the M_n value, with the applicable ϕ values from Section 2.3.1.3 (b) or (c), shall be used in Eq. (2-14) to determine the allowable uniform load for flexure, W_{utr} . When $KK_o < 1.0$, the V_n value and the ϕ factor from Section 2.3.1.3 (a) shall be used in Eq. (2-10) to establish the permissible superimposed uniform live load for shear-bond, W_{ts} . The nominal shear strength, V_n , is:

$$V_n = 12 (M_n)/\ell'_i \quad (D-20)$$

(This Commentary is not a part of the Standard. It is included for information purposes.)

Commentary on ASCE Standard for the Structural Design of Composite Slabs

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(This Commentary is not a part of the Standard. It is included for information purposes.)

Commentary on ASCE Standard for the Structural Design of Composite Slabs

CHAPTER 1 - GENERAL

1.1 - Introduction

1.1.1 - Composite slabs. Cold-formed steel deck sections are used in many composite floor slab applications. In such applications, the steel deck serves not only as the form for the concrete during construction but also as the main tensile reinforcement for the slab. The term "composite slab" is applied to floor systems where the steel deck has some mechanical means of providing positive interlocking between the deck and the concrete. Two examples of composite floor slabs are given in Figures C1.1¹ and C1.2. The system shown in Figure C1.1 illustrates composite action by means of embossments or indentations in the plate sections of the steel deck. The composite action shown in Figure C1.2 is achieved by means of transverse wires welded to the top horizontal surfaces of the steel deck. In addition, composite action may be achieved between the composite steel deck floor slab and the support beams by attaching studs or approved effective shear connectors, as shown in Figure C1.2.

In some instances, the steel deck may be considered only as a form for the concrete, in which case the deck and the concrete are not assumed to act compositely. For this noncomposite application, the deck is designed to carry the construction loads, deck

dead load, and wet concrete dead load, prior to curing of the concrete. After curing, the deck usually is not counted on for slab reinforcing; rather a conventionally reinforced concrete slab is designed to carry the applied loading.

However, most steel deck applications achieve the greatest economy when the steel deck can provide composite action by interlocking with the concrete.

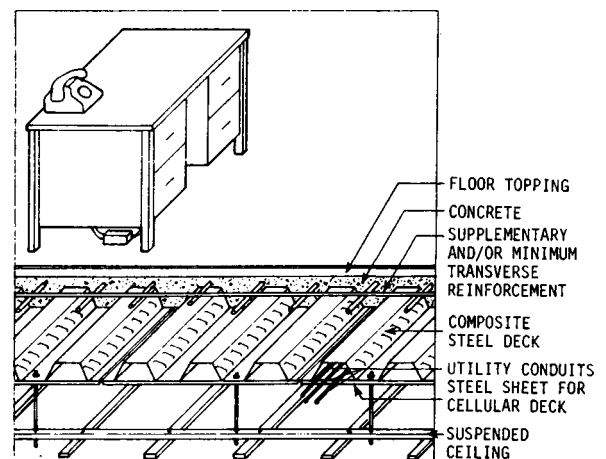


Fig. C1.1 - Typical building floor construction utilizing cold-formed steel deck

¹ References, figures, and equations with numbers beginning with "C" appear in this document. Citations of other section, equation, table, and figure numbers refer to the Standard² document; cross references to sections within the Commentary are clearly indicated. Each section number of this Commentary corresponds to the same numbered section of the Standard².

² Unless otherwise noted, The "Standard" or "design Standard" referred to in this Commentary is the document entitled "Standard for the Design of Composite Slabs" [C.46], and the "Standard Practice" is the document entitled "Standard Practice for Construction and Inspection of Composite Slabs" [C.36]³.

³ Numbers in brackets denote REFERENCES listed after Chapter 3.

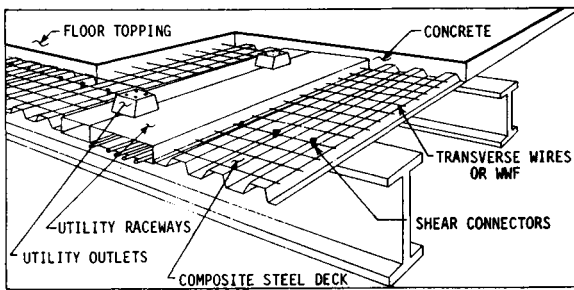


Fig. C1.2. - Typical building floor construction utilizing cold-formed steel deck with composite support beams.

The Standard addresses these composite applications.

In addition to acting as a form for the concrete and as the main tensile reinforcement (in positive bending), the steel deck may also (1) serve as a working platform for the workmen, materials, and equipment during construction; (2) provide support for a suspended ceiling as shown in Figure C1.1 or serve as the ceiling surface itself; and (3) provide raceways as shown in Figure C1.2 for utility distribution; and (4) provide lateral bracing of steel beam flanges.

Composite slabs utilize steel decks having some type of mechanical interlocking with the concrete fill. Steel decks not having any type of mechanical interlocking devices shall not be used for composite slab design as defined herein. This mechanical interlocking is provided by shear transfer devices or by the deck's profile geometry or by a combination of shear devices and deck profile. The mechanical and/or profile interlocking of composite steel decks must provide for resistance to vertical separation and to horizontal slippage between the deck and concrete surfaces. Surface bonding by itself (e.g., smooth decks without mechanical interlocking devices) is not considered sufficient for composite design. Examples of shear transfer devices are embossments (vertical horizontal, inclined, or combinations of such directions), transverse wires, welded buttons or washers, and holes.

In current practice, the shear transfer devices on a particular deck have fixed, uniform spacings; that is, all embossments, wires, welded buttons, or holes are

equidistantly spaced along a single deck sheet. For such decks where spacing is the same for all sections of the same basic profile, the spacing is not considered as a design parameter. However, the spacing of the devices need not be the same from one deck application to another. Some deck manufacturers make decks with identical profiles but varying spacings of the shear transfer devices. Those decks in which the spacing of shear transfer devices may be changed from sheet to sheet are said to have varying spacing of the shear devices. For such decks account must be taken of the spacing in the performance tests and design procedures. Each deck with varying spacing is to be regarded as a separate, unique deck profile for testing purposes.

No current knowledge accounts for decks when spacing varies along a single sheet since this arrangement is not used in practice. However, Section 3.2 should be considered with the general intent of the other sections of the Standard if such a case should arise. For those cases where the spacing is uniform for each sheet but varies from one profile to another, the general provisions of the Standard can be applied. However, each spacing arrangement must be considered as a separate parameter in Sections 3.2 and 2.3.1.5.

Deck sections generally are classified as cellular or noncellular decks. See Figure C1.3. Cellular decks differ from noncellular decks in that the cellular deck profile has closed cells formed by an added sheet of steel connected to the bottom corrugations of the deck. The closed cells are used for electrical, communication, or other utility raceways within the floor system. Frequently cellular and noncellular composite deck units are blended together. See Figures C1.2 and C1.3.

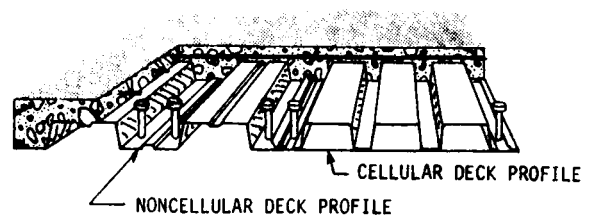


Fig. C1.3 - Illustration of a typical blended cellular and noncellular type of deck profile.

1.1.2 - Objective and scope. The requirements contained in the Standard are considered minimum recommendations. In some cases, construction and design conditions will dictate more stringent requirements than those in the Standard.

The design of steel deck floor slabs is based primarily upon the load-carrying characteristics in one direction, that is parallel to the deck corrugations. Other design considerations, such as the effects of concentrated loads, temperature, shrinkage, holes, etc., may necessitate consideration of load distribution in a direction transverse to the deck corrugations.

The primary load-carrying design method for the composite steel-deck-reinforced floor slab is based upon ultimate strength principles. The actual ultimate load-carrying capacity is verified by testing full-scale sections to failure (performance tests). These performance tests provide information about the behavioral characteristics, mode of failure, and load-carrying capacity. The results of the performance tests are used to obtain constants for the strength design equations or to verify the design equations.

The design method for the loading of the steel deck prior to curing of the concrete is based on accepted elastic principles. The design considerations for deck properties and allowable stresses under this stage of loading follow the American Iron and Steel Institute (AISI) Specification for the Design of Cold-Formed Steel Structural Members and related documents [C.1].¹ Additional information not contained in this Commentary but of interest to the designer is given by References [C.2]-[C.28]. Most of these references pertain to the findings from an extensive research testing program sponsored by the American Iron and Steel Institute and conducted at Iowa State University by C. E. Ekberg, M. L. Porter, and R. M. Schuster. Other references pertain to research on in-plane shear diaphragm action sponsored by the National Science Foundation [C.19-C.21, C.24-C.27, C.29-C.30]. Over 1,000 full-scale composite slab specimens were tested and the results analyzed, along with data compiled from over 150 specimens tested by various manufacturers.

This Commentary is not intended to provide a detailed summary of all the studies and research data

reviewed by the Steel Deck with Concrete Standards Committee in formulating the provisions of the Standard. However, references to some of the research data are provided for those who wish to review the background material. Several of the standards established by the American Society for Testing and Materials (ASTM) [C.31] are referred to in the Standard and Commentary.

The principles of reinforced concrete as applied to steel deck slabs follow the American Concrete Institute (ACI) Building Code Requirements for Reinforced Concrete ACI 318 [C.32]. Various standards of the ACI and other organizations are cited in the Standard and Commentary. Since these standards are revised from time to time, revised editions should be checked to ascertain appropriateness to other applicable codes and this Standard. In the event of any conflict with other code and standard documents, the design engineer or the building official having jurisdiction must determine the applicable governing sections.

The Standard is not intended to cover highway bridge load applications where the various specifications of the American Association of State and Highway Transportation Officials may apply. This does not imply that composite or noncomposite decks cannot be utilized for such applications.

1.1.3 - Inch-pound units. In inch-pound units, common usage is that loads are given as pounds per square feet and cross-sectional properties of steel decks are given based on a foot width of deck. These have been retained in the Standard for ease of the user. Similarly, in SI units, loads are given in kilonewtons per sq. meter and sectional properties are given per meter width of deck. The companion metric equations in parentheses are using these values in the Standard. SI units as given in parentheses and Appendix A are for information only and are not part of the Standard. Where separate metric equations are not shown, a consistent set of SI units can be used.

1.2 - Materials

1.2.1 - Steel deck. The more common grades of structural quality carbon and high-strength, low-alloy sheet and strip steel used for manufacturing cold-formed steel decks are ASTM A446, and A611 [C.31]. Other approved grades of steel are specified

28 ¹ Numbers in brackets denote REFERENCES listed after Chapter 3.

in the AISI Specification [C.1]. The minimum specified yield strengths of these steels range from 33 ksi (230 MPa) to 80 ksi (550 MPa).

Each steel deck manufacturer should specify the appropriate standard materials being used. Mechanical properties of the steel deck material are determined in accordance with the requirements of the AISI Specification [C.1] or the appropriate ASTM Standard [C.31].

Manufacturers of steel decks with welded transverse wires should indicate the ASTM Standard designation pertaining to the wires. For example, deformed wire is to conform to ASTM A496, and smooth wire is to conform to ASTM A82 [C.31].

Many steel decks are formed from zinc-coated, pretreated, or prepainted sheets (e. g. using iron phosphate or zinc-coating treatment). Zinc coating usually ranges from light G01 up to G90 galvanizing (Z01 up to Z90) (0.90 oz/ft² (90g/m²)) as specified by ASTM A525 [C.31] or Federal Specification QQ-S-775d [C.33]. Some applications may require heavier galvanizing. Past applications have included as high as 1.25 oz/ft² (125 g/m²). Some decks may have a phosphate treatment on the top side and a precoated or painted surface on the bottom side.

If the steel deck is exposed to a corrosive environment, the surface treatment should adequately protect the steel. The type of surface treatment must be specified by the designer. Heavy rusting of the deck should not be allowed to take place. However, light rust (tight rust), such as might occur prior to installation, generally will not affect the structural integrity or performance of the deck. This light rust is acceptable in normal applications where the deck will not be subjected to a corrosive environment or continued rusting after installation. The effects of this tight rust and other coatings were compared and the corresponding strengths and behavioral differences are given in Ref. C.22. See also Section 3.2.1.1 of this Commentary and Section 2.2.2 of the Standard Practice Commentary [C.34]. For additional recommendations on protective coatings refer to the AISI *Supplementary Information on the Specification for the Design of Cold-Formed Steel Structural Members* [C.1]. In general, the composite deck must be designed to last for the entire life of a structure since it serves as the primary positive

bending moment reinforcement. Thus, a minimum surface finish is usually required. When using a galvanized sheet in a corrosive environment, a minimum G60 galvanizing is suggested. Some applications will require a heavier surface protection. The surface condition of supplementary reinforcing steel must meet the requirements of Sections 3.5 and 7.4 of ACI 318 [C.32].

No minimum thickness of steel deck is stipulated in the Standard. However, considerations of design and construction will dictate a minimum cross sectional area and thickness for a particular deck profile. In general, the thickness, *t*, should not be less than 0.023 inches (0.58 mm). The minimum thickness of steel is usually stipulated in the job specifications.

1.2.2 - Concrete. The concrete for composite slabs shall be of the structural quality established by ACI in ACI 318, Building Code Requirements for Reinforced Concrete [C.32], and in several ACI standards mentioned in the Commentary to the 318 Code and in the Manual of Concrete Practice [C.32]. See also Sections 1.2.2 and 2.2.2 of the Standard Practice Commentary [C.34]. Generally, the other standards are considered a refinement of ACI 318; however, in the event of a conflict of the two, the design engineer shall determine which one prevails. Additional ACI standards that apply to composite slab construction are:

ACI 211.1	Standard Practice for Selection Proportions for Normal, Heavyweight, and Mass Concrete [C.32]
ACI 211.2	Standard Practice for Selecting Proportions for Structural Lightweight Concrete [C.32].
ACI 301	Specifications for Structural Concrete for Buildings [C.32]
ACI 302.1R	Guide for Concrete Floor and Slab Construction [C.32]
ACI 304R	Recommended Practice for Measuring, Mixing, Transporting, and Placing Concrete [C.32]
ACI 305R	Hot Weather Concreting [C.32]
ACI 306R	Cold Weather Concreting [C.32]
ACI 308	Standard Practice for Curing Concrete [C.32]
ACI 309R	Guide for Consolidation of Concrete [C.32]

ACI 318 or 318M	Building Code Requirements for Reinforced Concrete [C.32]
ACI 318.1 or 318.1M	Building Code Requirements for Structural Plain Concrete [C.32]

Composite slabs may be constructed with either normal, semi-light-weight or lightweight structural concrete. Relatively little research has been conducted to ascertain the difference in behavior between composite slab systems constructed with normal and with lightweight concrete. Each method of shear transfer is expected to be affected differently by the concrete type. Therefore, each manufacturer should obtain design loadings for each type of structural concrete in accordance with the performance testing program given in Chapter 3 of the Standard. For lightweight applications, only a structural grade of lightweight concrete as defined by the ACI standards [C.32] is permissible. (See particularly ACI 211.2).

ACI 318 [C.32] gives requirements for maximum size of aggregate in relation to the clearance dimensions and reinforcing bars (in the case of additional steel for negative moment reinforcing, etc.). The size limitations on aggregates are necessary to ensure proper encasement of the reinforcing bars and penetration into steel deck ribs and/or holes. In addition, aggregate gradation and proper concrete mix proportions are important to achieve the desired concrete consistency and strength.

For those sections using holes as shear transfer devices, the maximum aggregate size should be smaller than the hole. This prevents any possible plugging by a larger piece of aggregate restricting the proper flow of the concrete through the hole.

The deck profile itself may restrict the maximum size of the aggregate. For example, a deck section having corrugations close together may have the maximum aggregate size dictated by the spacing or geometry of the corrugations. This limited clearance should not occur in most decks because adequate distance between corrugations must also be provided for the welded shear connectors that may be used with supporting composite beams.

Aggregate size limitations may occur in relation to the minimum depth of concrete over the deck (see Section 1.3.2 of the Standard). Additional aggregate size limitations with regard to proper clearances

between reinforcing steel placed over the deck and the top of the deck should be in accordance with accepted standard concrete practices and the **Standard Practice for Construction and Inspection of Composite Slabs** [C.36].

Chloride salts are detrimental because the electrolytic action resulting from the presence of salt may cause corrosion of the steel. Thus, the Standard prohibits the use of admixtures containing chloride salts. In addition, the chloride ion content of the mixing water and the aggregate should not be excessive. Section 4.3 of ACI 318 [C.32] gives limitations for chloride ion content. Section 3.4.2 of the 1977 edition of the ACI 318 Commentary states that a chloride ion content of 400 ppm or 500 ppm might be considered dangerous for applications involving prestressed concrete or aluminum embedments and suggests that levels well below those values be maintained if practical. The ACI 201 Guide to Durable Concrete [C.32] suggests limits that are slightly more restrictive than ACI 318. Though not strictly applicable to composite steel deck slab construction, these recommendations are included for general guidance.

1.3 - Slab limitations

1.3.1 - Span-to-depth ratios. The span-to-depth limitations given in the Standard are based on industry-wide experience and commonly accepted practice. These maximum ratios provide guidance for obtaining satisfactory deflection limitations under static service load conditions for members not supporting or attached to partitions or other construction likely to be damaged by large deflections. However, the designer should compute deflections and compare them to the allowable deflection criteria contained in Section 2.3.2.2 of the Standard. The span-to-depth limitations are not intended as an alternative to the provisions of Section 2.3.2.2. The span-to-depth ratios given in the Standard are slightly higher than those for conventional reinforced concrete [C.32]. The justification for this stems from the fact that a composite deck slab has steel on the outside of the concrete instead of embedded in the concrete as in a conventional reinforced concrete system. For structural supporting elements likely to be damaged by large deflections, a greater slab stiffness than that provided by the given ratios may be necessary.

1.3.2 - Slab depth and concrete cover. The minimum slab depth of 3 1/2 in. (90mm) is based on a 1 1/2 in. (38 mm) deep deck and a minimum of a 2 inch (50 mm) cover over the top plane of the deck. This depth is considered to be the minimum needed to provide for adequate cover, placement of supplemental steel for shrinkage and temperature requirements, and stiffness of the slabs.

Slabs may be made to act compositely with the support beams by means of shear studs welded to these beams through the deck. Slabs may require a greater slab depth in order to provide the necessary concrete thickness for the studs as prescribed by the American Institute of Steel Construction (AISC) Specifications [C.37] or to achieve a specified fire rating [C.38] where required by the building code or engineer.

The minimum cover for the reinforcement (steel deck or otherwise) is measured from the top fiber of the concrete to the top fiber of the reinforcing steel. When concrete surface finish fluctuations are apt to exist, or if the top of the concrete is subject to wear and abrasion or is in contact with the environment, then additional cover may be required. The provisions of Section 7.7 of ACI 318 [C.32] serve as additional guides.

In some cases the minimum cover over the steel deck may be controlled by considerations other than structural. These may include noise and heat transmission and certain factors documented in fire and electrical codes. Formwork criteria for the subdivision of floor sections, holes, and floor edges should comply with Chapter 6 of ACI 318 [C.32].

1.4 - Tolerances and minimum sizes

1.4.1 - Tolerances. The guidelines for tolerances given in Section 1.3 of the Standard Practice document indicate the maximum variation allowable between the deck utilized in the tests and that furnished in practice. The negative percent defines how much the furnished deck can be less than the tested deck for a given category. In addition, the properties of decks used in design shall not vary more than those of the tested deck by the tolerance amounts stated. For example, a -5% tolerance on depth means that the depth of the embossment on the furnished deck may be as much as 5% shallower than

the designed (by Chapter 2 of the Standard) or tested (by Chapter 3) steel deck section but any amount deeper. The designer utilizing various shear transfer devices may wish to consider a minimum size or spacing; e.g. in the case of embossments, a minimum depth of embossment should be specified. Tolerances for the depth of the slab, the location of negative moment reinforcement, and the location and style of supplemental reinforcement not given by the Standard are governed by the standard tolerance guidelines in Sections 7.5 and 7.6 of ACI 318 [C.32]. The tolerance given for welds utilized for shear transfer devices should also include a proper testing of the welded device along with the tested deck in Chapter 3.

Additional tolerances beyond those listed in the Standard Practice may be applicable. As a guide, the deck supplied by the manufacturer to the job site should not vary from the tested deck beyond the following:

- (1) Overall deck depth should not be less than 1/32 inch (0.8 mm) from that tested.
- (2) The width of flat elements should not change by more than + 1/16 inch (1.6 mm) or - 1/32 inch (0.8 mm).
- (3) The cross-sectional area of the steel deck should not result in a decrease of more than 5% from that tested (a furnished larger area is acceptable).
- (4) The radius of the curved elements should not change by more than + 10%.
- (5) The Specifications indicate a $\pm 1/4$ inch (6 mm) tolerance on the spacing of shear devices. This tolerance should not exceed $\pm 10\%$ of the individual spacing.

In some cases the ACI 117 document on Standard Tolerances may be useful [C.32].

1.4.2 - Dimensions. Since the strength of the tested specimens is used to determine design coefficients, the dimensions of the steel deck, including mechanical shear transfer devices, should be tested within the tolerances of Sections 1.4.1. and 1.3 of the Standard Practice [C.36].

CHAPTER 2 - DESIGN CRITERIA

2.2 - Construction stage

2.2.1 - General. This section pertains to the steel deck prior to any composite action. The deflection of the deck under the weight of wet concrete usually results in a variable depth of concrete. This is because the top of the slab is normally finished at a level surface; consequently, the depth of the concrete is greatest at the point of largest deck deflection. For short spans, the increased weight with deflection may be quite small but significant with longer spans. Due consideration must be given to deflection related weight increases in computations for deflections, stresses, deck deflection, or the increased volume of concrete required.

An approximate solution to the problem of computing the increased weight due to deflection is to assume that the deflected shape is parabolic. The volume of the additional concrete due to deflection can then be taken as $(2/3)\Delta_i \ell_f B$ for a simple span, where Δ_i is the uniform load slab deflection, B is the slab width, and ℓ_f is the span. The volume is in terms of the units on Δ_i , ℓ_f , and B . A new Δ_i then can be approximated taking into account the increases in concrete volume. The increases in volume also can be influenced by deflections in the supporting structure which must be evaluated.

2.2.2 - Section properties. The properties discussed here apply to construction loads placed directly on the deck where the deck is required to act as the load-carrying member. The principal consideration is the determination of the effective moment of inertia of the deck section only, which depends on the effective width of the elements in compression. The effective width is a function of the width-to-thickness ratio and the compressive stress. The effective width of the deck section may be increased by the use of intermediate stiffeners. Attention is called to the design aids published in Part V, Charts and Tables, of the AISI design manual [C.1]. The uncoated thickness is used for design purposes and for evaluating the results of the performance tests.

Depending on the configuration and thickness of the deck, different effective moments of inertia (and section moduli) may exist for the steel deck when it is subject to positive bending as opposed to negative bending. Steel decks that are continuous or that utilize shoring will require investigation using the

moments of inertia for positive and negative bending. In addition, the moment of inertia used for deflection calculations may be different from that used for stress calculations. See Part II, Commentary, of the AISI design manual [C.1].

Section properties are based on calculations utilizing a complete steel deck panel including side lap joint segments. The section properties are published by manufacturers on a per-foot-of-width basis. (For SI units, a per-meter-of-width basis may be more appropriate.)

The load-carrying capacity of decks that contain perforations of significant size in the cross section is not covered in the AISI Specification for the Design of Cold-formed Steel Structural Members, Sections B, C, D and E [C.1]. Section F of that Specification states the tests to be conducted and the method of evaluating them to determine the structural performance of the steel deck.

2.2.3 - Construction live loads. The 20 psf (1.0 kN/m²) uniform and the 150-lb-per-foot-of-width (2.2 kN/m) concentrated construction live loads given in the Standard are each considered separately rather than simultaneously and are added to the concrete and the steel deck dead loads for determination of the maximum steel stress in the deck. The 150 lb (2.2 kN/m) live load is usually taken transverse to the deck corrugations (on a per-foot-of-width or per-meter-of-width¹ basis). The stated construction live loads are considered minimum loads applied to the steel deck during the construction stages. If higher loads exist (such as from transient or other sources), those load values listed in the Standard should be appropriately modified. Special consideration must be given to high-magnitude concentrated loads or moving equipment where temporary planking may be required to avoid local damage to the deck. Sections 2.2 and 2.1.5 of the Standard Practice provide for planking for highly concentrated transient loads.

The loading diagrams of Figure 2.1 consider the sequence of construction for shored and continuous spans for up to three spans. Similar determinations are needed for a higher multiple of continuous spans. Impact loading should be avoided during construction. Loading sequences other than those shown in Figure 2.1 may need to be considered, for example, those due to pattern loadings if existent.

32 ¹ Unit width of slab in SI units shall be consistent with SI units for other terms.

Stress and deflection calculations should consider the sequence of construction for all shored and continuous spans. See also Standard Practice Commentary Section 2.2.2.

2.2.4 - Allowable stresses. In addition to the usual limit for bending stress of $0.6 f_y$, an upper limit of 36 ksi (250 MPa) is used for the maximum allowable stress. The 36 ksi controls for the higher strength steels (e.g., ASTM A446, Grade E [C.31]) and provides some check on required ductility.

2.2.5 - Calculated stresses. Prior to the development of composite action, the maximum steel stresses due to bending are determined from elastic analysis using the conventional relationship

$$f_b = 12 M/S \left(= \frac{M}{S} \right)^1 \quad (\text{C.1})$$

where

- f_b = bending, stress for elastic computations, psi (MPa)
- M = moment due to construction live load, concrete dead load, and steel deck dead load, ft-lbs/ft of width (Nm/m)
- S = appropriate effective section modulus for either positive or negative bending, in.³/ft of width ((mm)³/m)¹

Maximum values of M (for applicable cases) are shown in Figure 2.1.

For shored and continuous span systems, the maximum steel stress is determined for both positive and negative bending moments by using the appropriate effective section modulus as discussed in Section 2.2.2 of the Commentary.

In addition to the bending stress, shear stress and web crippling of the deck should be checked in accordance with Sections C.3.2 and C.3.3 and Sections C.3.4 and C.3.5, respectively, of the AISI Specification [C.1]. See also Commentary Sections 2.2.2 and 2.3.1. Computation of these stresses may be made by considering the sum of the gross areas of the web elements in a given width, normally 1 ft. (305 mm).

Proper bearing to prevent crippling of the web portions of the deck must be considered in all deck systems. Sections C3.4 and C3.5 of the AISI

Specification [C.1] give criteria for determining adequate bearing. Generally, the bearing support length should not be less than 2 inches (50mm) but usually will be substantially greater.

2.2.6 - Dead load deflection. The deflection limitation applies to continuous and shored slabs as well as simple span deck systems. Deflections are to be computed by the appropriate deflection relationships for simple or continuous spans, such as those shown in Figure 2.1. Loading sequences, other than those shown in Figure 2.1, may need to be considered, if they exist. In some instances, increased deflections due to the ponding effects of the increased weight of concrete and deflections of the deck and its supporting members must be considered. See Commentary Section 2.2.1. The length ℓ_{nr} is taken as the clear span between temporary or permanent supports for deflection calculations provided that adequate deck tie down is achieved on the primary support members. Thus, $12\ell_{nr}$ would be used in place of ℓ_i in Figure 2.3 when adequate tie down exists so that the point of "pin" support can be considered at the edge of the support beam.

2.3 - Composite section

2.3.1 - Strength design

2.3.1.1 - General. Composite strength design pertains to calculation of allowable live and dead loads that can be supported safely by the composite steel deck slab. The provisions of Standard Section 2.3 provide the basis for strength and serviceability designs utilizing steel decking as reinforcement for composite slab systems. Normally, the composite slab is designed so that the steel deck alone constitutes sufficient reinforcement to resist positive bending moments due to service loads. However, additional reinforcing steel may be needed for negative bending, temperature and shrinkage reinforcement, floor openings, or two-way slab action. Design for superimposed live loading is found from the calculation of strength based on shear-bond and bending. Usually, the shear-bond strength will be governing, although possible bending failures can occur in either an under- or overreinforced mode. For cantilever spans, the steel deck acts principally as a form for the concrete; therefore, top reinforcing must be designed.

¹ Unit width of slab in SI units shall be consistent with SI units for other terms.

2.3.1.2 - Load factors. The load factors listed in this section of the Standard were taken from ASCE 7 (Formerly ANSI A58.1) [C.39] for strength design concepts. Due regard should be given to the directions of the dead, live, wind, and earthquake load effects. Combinations of forces in opposite directions could cause reverse bending and necessitate added reinforcement in the top of the floor slab. This reverse loading would be particularly true for pressure or blast load occurring from the underside of the floor. Because of the improbability of full dead, live, and wind (or earthquake) loads all occurring simultaneously, 75 % of this load combination is used for the ultimate uniform load. Equations (2-1) through (2-6) in the Standard give the required factored design loads in terms of a uniform load, W_u^1 . This is done because most design floor loads are uniform.

If impact effects are present, such as may be the case for parking buildings or warehouse floors, impact loads are usually considered with live loads in Eqs. (2-2), (2-3), (2-5) and (2-6); however, impact or dynamic load considerations should also be included in wind and earthquake loads in Eqs. (2-2) through (2-6). The load effects due to differential settlement, creep, shrinkage, and temperature should also be included with dead loads in Eqs. (2-1) through (2-6). Special applications involving nuclear structures may require a lower probability of potential failure and thus lower strength reduction or higher load factors.

2.3.1.3 - Strength reduction factors, ϕ . The strength reduction factors contained in this section differ somewhat from those of the ACI 318 Code [C.32]. The factors vary according to the mode of failure and are chosen on the basis of behavioral characteristics that occur prior to failure. Variation in laboratory parameters such as embossment depth and other shear device tolerance variations will affect the strength. The majority of the shear-bond failures experienced in laboratory tests have occurred suddenly, without ample warning of impending failure. Therefore, a strength reduction factor, ϕ , equal to 0.75 was chosen for the shear-bond failure mode, somewhat lower than for shear failure in conventional concrete design.

Underreinforced flexural failures are usually preceded by noticeable deformations and cracking (i.e. more ductility than the shear-bond mode), and thus ϕ =

0.85 is used in accordance with the load factors used in the ASCE 7. However, when the steel deck has relatively low ductility ($f_u/f_y \leq 1.08$ based on minimum specified tensile and yield strengths), ϕ is reduced to 0.65 because of the possibility of a sudden failure due to an abrupt tearing of the bottom steel plate elements.

Similarly, overreinforced flexural members depend more directly on variations of the concrete strength and may or may not give ample warning of impending failure; thus, overreinforced slabs are assigned a ϕ value of 0.70. This situation is somewhat similar in behavior to that in reinforced concrete columns. Only a few flexural failures of overreinforced slabs have been found in laboratory tests to date. The ϕ factor for bending in plain concrete, 0.60, is the same as in the ACI 318 Code [C.32], accounting for the changes in load factors.

Excessive bearing forces on the composite section can cause parallel splitting (parallel to the deck corrugations). The ϕ factor for bearing in this case is not given. If high bearing forces are suspected, tests are suggested and an appropriate ϕ factor for the observed splitting or bearing behavior should be selected. If bearing causes a tensile splitting, a ϕ factor not greater than 0.80 is recommended.

2.3.1.4 - Continuity over supports. The steel deck is normally placed to span three or more supports. This placement is not intended to ensure an uncracked section over the supports. Therefore, steel deck floor slabs are designed as simple span, one-way systems unless conventional negative bending moment reinforcement is provided.

For cases where the designer wishes to consider the floor system as a continuous slab, elastic analyses, or coefficients, such as those in the ACI 318 Code [C.32] may be used. The design moments and shears for simple or continuous span systems are obtained using the factored loads contained in Section 2.3.1.2 of the Standard. The designer should not use the approximate coefficients in Section 8.3.3 of the ACI 318 Code unless the stated assumptions for their use are satisfied. Use of the coefficients assumes that the design is for a one-way slab spanning parallel to the deck corrugations and contains negative moment reinforcing.

34 ¹ Unit width of slab in SI units shall be consistent with SI units for other terms.

When slabs act as independent simple spans, the rotations over the supports may require nominal reinforcement for control of crack widths. This reinforcement should be kept near the surface (3/4 in. (20 mm) cover) of the slab for crack control but is normally not considered to provide continuity for negative bending moments. Nominal reinforcement over the supports usually consists of small reinforcing bars, smooth wire, or deformed wire and is frequently not accounted for in the strength analysis of the section. Small, hairline type cracks in the top of the slab over supports will not affect the shear-bond strength or service behavior of the slab.

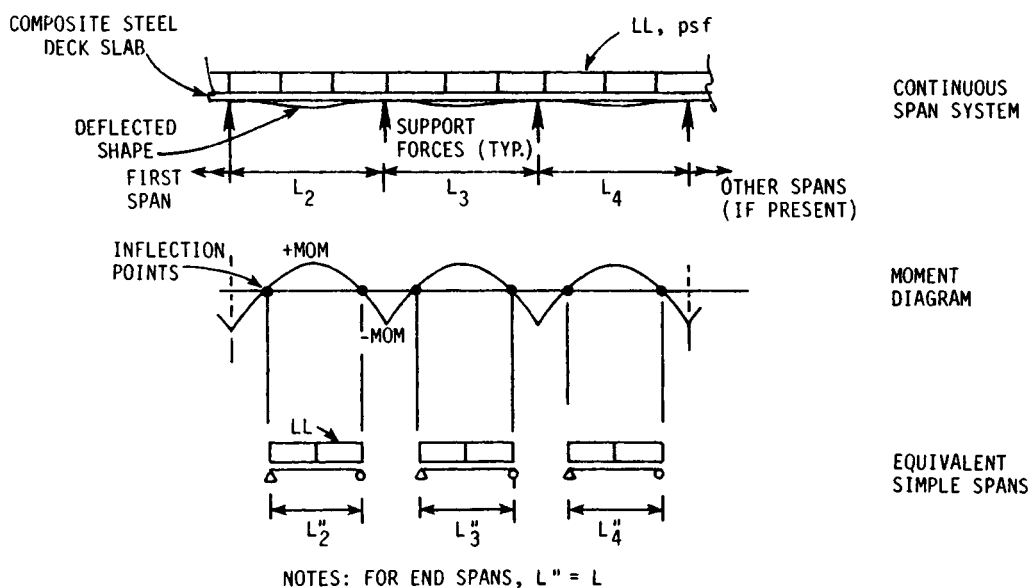
If less cracking over the supports is desired, a continuous slab design by furnishing adequate steel for negative bending may be advantageous. In such cases, the designer may wish to use a strength design of the cross section for conventional negative bending reinforcement. If welded wire fabric (WWF) is used as the crack control reinforcement, the designer may require an additional strip or area of WWF to be placed through the joint. Those cases involving composite shear studs may require proper crack control to ensure composite beam-to-slab action.

Special consideration must be given to the location of construction joints. Care is needed to ensure that structural continuity and crack control exist. In addition, the location of the construction joint should not interfere with composite slab-to-support anchor for those cases involving studs or other composite shear connectors.

If a conventional reinforced continuous slab over interior supports is designed, the design engineer must consider the nonrectangular cross section in the compressive zone under negative bending. However, the beneficial effect of the steel deck in compression can be neglected as per the designer's option.

A question arises as to the proper span length to be used for shear-bond design determinations for continuous spans where adequate negative reinforcing is provided. Few tests have been conducted on continuous span systems. Therefore, no definite design recommendations regarding the proper shear-bond span length can be made. However, the following serves as a guide.

Figure C2.1 illustrates a typical continuous span, one-way floor system subjected to a uniform live



Notes:

1. For end spans, $l''_i = l_i$.
2. Use W_i on a per ft. length and per ft. width section of deck.

Fig. C2.1 - Continuous span system indicating equivalent simple spans, l''_i

load. The spans in Figure C2.1 are assumed to be nearly equal and generally should meet the requirements of Section 8.3.3 of the ACI 318 Code [C.32]. The sign convention of positive and negative moments mentioned in the Standard follows the "+" and "-" marking in Figure C 2.1.

Shown in this figure are equivalent simple span segments of length ℓ''_i . The length ℓ''_i is the distance between inflection points in each span. These equivalent simple span segments are used in determining the shear-bond strength only, not in the flexural design. Slabs subjected to concentrated loads generally will require a special ℓ''_i determination or, if conservative, use of the full span length, ℓ_i .

Shear-bond failure is considered only for that portion of each span subjected to positive bending under uniform loading.

The ℓ''_i lengths may be used in place of ℓ_i for each interior continuous span in the shear-bond Eq. (2-9) in Section 2.3.1.5.1. of the Standard. See Section 3.2.2 of the Commentary for an explanation of the shear-bond type failure. Note, however, that γ must be adjusted to reflect this substitution for interior spans shored at the center. See Section 2.3.1.5.1. of the Commentary for a discussion of γ .

The shear at the end of the equivalent simple span segment is, of course lower than the maximum shear at the interior supports. The use of the higher design shear appears reasonable for the positive bending in interior spans since shear-bond horizontal slippage of the concrete is prevented by the adjacent spans, thereby providing blocking action which aids in preventing a potential shear-bond failure. The end spans, however, do not have the adjacent span on the exterior end; thus, the full exterior span length should be used for shear-bond determination, since shear-bond slippage could occur over the region in the vicinity of the exterior support when little or no end restraint exists. Thus, no beneficial blocking action occurs for the exterior span of a continuous system unless provided by installed items such as concrete beams or studs.

2.3.1.5 - Strength relationships. The strength relationships 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 concentrated loads such as fork-lift trucks. For slabs subjected to concentrated loads, special consideration must be given to the ℓ'_i in Eq. (2-8), and the transverse distribution of loads must be considered. See also Sections 2.3.3.2 and 2.3.3.3 (Standard and Commentary) and Reference [C.15].

The designer should be aware that shear-bond strength and bending are not always the controlling modes for design. Other controlling strengths may also exist, e.g., punching shear, bending in a transverse direction for those cases involving large, highly concentrated loads or holes in the slab, or bearing. Slabs containing blends of cellular and noncellular deck may require special considerations of strength.

2.3.1.5.1 - Shear-bond strength. The predicted shear-bond strength, V_u , based on one-way slab action subjected to a two-point concentrated line loading is given in Eq. (2-8) of the Standard. Extensive test results, described in References [C.2] through [C.18], [C.22], and [C.28], were used to develop Eq. (2-8) and to provide behavioral information. This equation development is similar to that of Eq. (11-6) of the ACI 318 Code [C.32]. See Section 3.2.2 of the Commentary for a description of the shear-bond failure mechanism and Section 3.2.4 of the Standard and Commentary for the derivation of Eq. (2-8). For uniformly distributed design loads, Eq. (2-9) applies.

The constants m and k in Eqs. (2-8) and (2-9) are determined by an analysis of the results of an adequate number of performance tests. The constants are the slope and intercept, respectively, of the assumed linear relationship

between $V_u/bd\sqrt{f'_c}$ and $\rho d/\sqrt{f'_c}$ as shown in Figure

3.2.¹ The maximum experimental shear at failure obtained from laboratory tests is V_e . The weight of the slab is not included in V_e , since only a portion of the slab weight may contribute to the shear between the steel deck and the concrete. The factor γ accounts for partial slab weight and is discussed below. Chapter 3 of the Standard contains recommendations for conducting the performance test program to arrive at the values of m and k .

The shear-bond equations [Eqs. (2-8) and (2-9)] are for determination of shear strength at the end of a one-way element with corrugations parallel to the span. Shear-bond action in the manner represented by these equations does not occur transverse to the corrugations. Shear in the transverse direction normally does not control unless concentrated loading or other special conditions are present. See Sections 2.3.3.2 and 2.3.3.3 of the Standard, the ACI 318 Code [C.32] and References [C.7] [C.14] and [C.15] for conventional shear above the corrugations if the transverse direction is being considered.

In Eq. (2-8), the shear-span, ℓ' , is the distance from the reaction to the concentrated load in a two-point loading system, as shown in Figure C2.2(b). For uniformly loaded test systems, ℓ' can be taken as $\ell_i/4$ or $12\ell_f/4$, one quarter of the span length. The distance $\ell_i/4$, which is found by equating the areas of the shear diagrams for the concentrated and uniform load cases, was verified by test results [C.28]. Figure C2.2 shows shear diagrams for these two cases. The area of the shear diagram in Case (a) of Figure C2.2 is $V_u \ell_i/4$, whereas the area of the shear diagram in case (b) is $V_u \ell'$. Equating Cases (a) and (b) for the same total applied load yields $\ell' = \ell_i/4$, where $12\ell_f = \ell_i$.

The two cases have equal end shears and midspan

moments even though the shear gradients are different. A comparison of the corresponding midspan deflections shows that the deflection for the concentrated load case is only 10% greater.

In actual design situations, combinations of uniform and concentrated loads may require special attention for the proper selection of the ℓ' distance. If several concentrated loads exist, the designer may elect to treat the system as an equivalent uniformly loaded beam. In certain instances, the loading combination may require tests to determine the appropriate ℓ' for use in Eq. (2-8).

The term $\gamma W_s \ell_f/2$ in Eqs. (2-8) and (2-9) accounts for the amount of the self-weight of the slab carried by the floor system in composite action, i.e., the amount applied to the composite system after shore removal. Table 2.1 gives values for γ for various shoring conditions.

The "complete support" condition means that the steel deck is uniformly supported during casting along its entire length; thus, the deck is not carrying any dead load during construction. Therefore, upon removal of the complete support, all of the dead load is carried by the composite section. The "complete support" condition usually applies only to specimens cast in the laboratory.

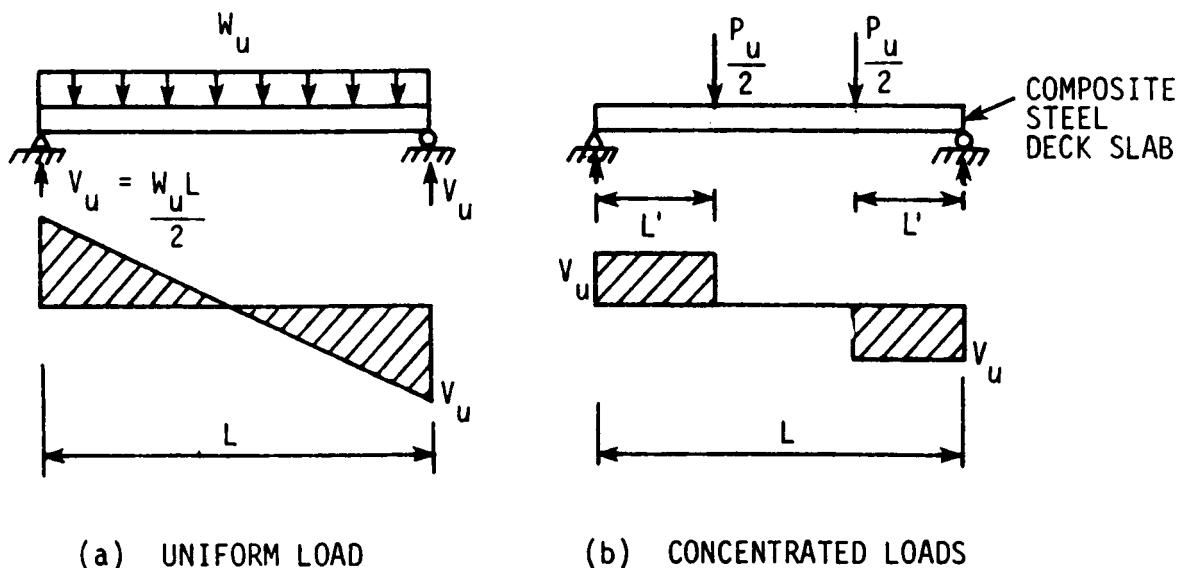


Fig. C2.2. Uniform and concentrated load applications

The opposite case is that of the unshored deck. For this case, the steel deck carries all of the dead load during casting, and no slab dead load is carried compositely; i.e., no interfacial shear forces are assumed to exist because of the dead load. For this case, only the dead load added after the concrete hardens, such as a hung ceiling, is carried compositely.

The case of the slab shored at the center is illustrated in Figure C.2.3. During casting, part of the dead load is carried by the shore support. When the shore is removed, the maximum shear added to the composite section is $(5/8)(W_s \ell_f/2)$, and so $\gamma = 5/8 = 0.625$ as given in Table 2.1.

The four most common support conditions are given in Table 2.1 as a means of illustrating the determination of γ . For cases involving other shoring support conditions, the additional γ factors should be determined in a manner similar to that shown in Figure C.2.3. Furthermore, the γ factors given are for shore conditions utilizing a simple span system. If continuity exists for composite action, then the ℓ'' distance discussed in Section 2.3.1.4 may be used in place of ℓ_f , thus converting the continuous system to an equivalent simple span system. The ℓ'' distance is not used to determine the γ factor for shoring. This equivalent simple span procedure is for shear-bond determination only and not for other failure mode determinations.

If, during the performance test evaluation, the experimental shears are corrected by the amount of dead load acting on the composite section, then Eqs. (2-8) and (2-9) should not include the $\gamma W_s \ell_f/2$ term. In other words, V_e is replaced by $(V_e + \gamma W_s \ell_f/2)$ in the plotting of Figure 3.2 of the Standard and in the modification of the shear-bond equations. With this technique, the constants m and k in Eqs. (2-8) and (2-9) include the shoring correction. If this technique is used, care must be taken to ensure that both shored and unshored conditions are used with all ℓ_f lengths.

Equations (2-8) and (2-9) do not account for the spacing of the shear transfer devices, where such devices are considered as variable. Therefore, separate m and k regression determinations are needed for each "s" spacing utilized.

2.3.1.5.2 - Flexural strength.

Equation (2-11)

Equation (2-11) defines the balanced steel ratio, ρ_b , for a steel deck slab with no compression reinforcement. This equation is a form of the conventional ACI equation [C.32], altered by the addition of the term $(h - d_d)/d$. The quantity $(h - d_d)/d$ in Eq. (2-11) accounts for yielding across the entire deck cross section when the concrete strain reaches 0.003. Note that E_s is the modulus of

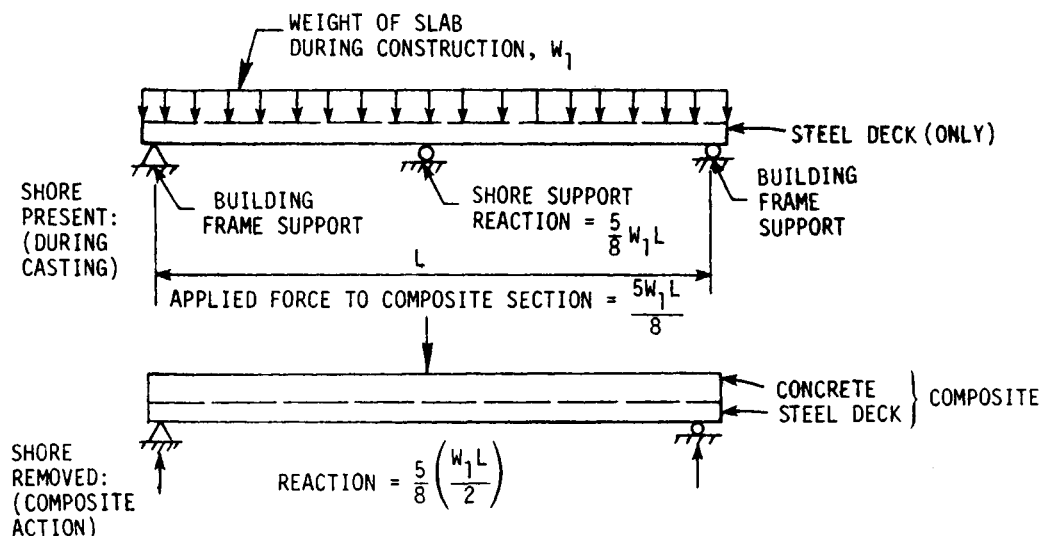


Fig. C2.3 - Illustration of γ for the case of single shore at center of simple span

elasticity of steel for cold-formed members instead of that for hot-rolled shapes.

Equation (2-11) is developed from the compatibility of strains together with the equilibrium of internal forces. For this development, the steel force is assumed to act at the centroid (c.g.s.) of the cross-sectional area of the deck. The development of Equation (2-11) assumes that no other reinforcing steel is present. This equation is valid only if the entire steel deck section yields when the maximum concrete strain is 0.003. Strain compatibility should be checked to assure that this assumption holds.

Equation (2-13)

Equation (2-13) applies to the underreinforced case where the entire deck section yields prior to the concrete reaching a strain of 0.003. This equation also assumes that the force in the steel acts at the deck c.g.s. and that no additional reinforcement is present (or is to be counted on for positive bending).

Equation (2-13) gives the calculated moment capacity on a cross section perpendicular to the steel deck corrugations. It is identical to the equation used in reinforced concrete design [C.32].

The designer must be aware that Eq. (2-13) is not valid unless there is yielding across the entire deck depth. There is a possibility that a very deep deck having a composite neutral axis within the deck profile may not reach yield at the top fiber at the ultimate flexural strength. Also, a deck made with steel having a low ductility (generally those grades of steel with $f_u/f_y \leq 1.08$ based on minimum specified tensile and yield strengths) may tear or fracture before the concrete achieves a strain of 0.003 or before yielding occurs across the entire deck section. One grade of steel where ductility may need to be checked to ascertain whether Eq. (2-13) is valid (i.e., can achieve the necessary tensile strain at the bottom fiber) is ASTM A446 [C.31], Grade E, having a minimum f_y of 80 ksi (550 MPa). In certain cases involving deep deck sections with a shallow cover, the top fiber of the deck cross section may buckle before achieving full flexural capacity given by Eq. (2-13). These situations are in no way detrimental to the particular steel deck system. They only dictate that a more general flexural strain analysis be used

which considers compatibility of strains together with equilibrium. A general flexural strain analysis procedure is discussed below in the next several paragraphs and depicted in Figure C.2.4.

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 Eq. (2-13) are not met and for cases of overreinforced sections, as defined in Section 2.3.1.5.2 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:
 - (i) rupture (tearing) of the bottom steel fibers
 - (ii) exceeding the maximum concrete compressive force (crushing of concrete)
 - (iii) buckling of the top fiber of the steel deck cross section (if in 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 (as per cold-formed design specification [C.1])
- (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} , shown in Figure C2.4. 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 Grade E of ASTM A446 [C.31], 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 0.005 for Grade E A446 steel may be selected and a strain of about 20-40 times f_y/E_s may be chosen for most other grades 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 strain in the top fiber of 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 .

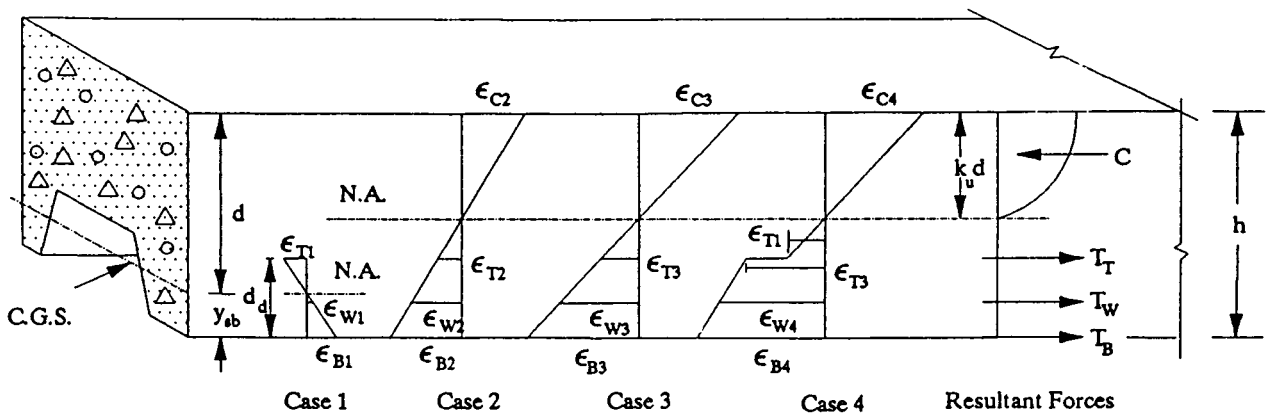
The engineer should exercise discretion when employing the general strain analysis to ensure that the proper selection of the controlling strain has been made, particularly for those instances where the deck is sufficiently deep to prevent yielding across the entire steel area. Special considerations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

General strain analysis can be used for instances when Eq. (2-13) cannot properly be used because the basic assumptions for this equation are not met. This analysis also applies to overreinforced slab elements.

Figure C2.4 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.2.4 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.2.4 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.2.4, 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.

Fig. C2.4 - Strain diagrams used to obtain general strain computed flexural capacity of 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.2.4). 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.

Cold-formed steel decking permitted by the AISI Specification [C.1] will usually have adequate ductility for yielding to occur over the entire cross section, except possibly decking made from Grade E 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 in two ways:

- (1) Ignore the supplementary steel and use Eq. (2-13) or the general strain analysis.
- (2) Use the general strain flexural capacity technique to account for the forces in addition to those of the steel deck.

The area of steel deck in Eq. (2-13), A_s , is generally intended to include only that portion (the cross section perpendicular to the corrugations) that is in tension and capable of achieving its yield strength.

A strain compatibility technique is needed for cases involving overreinforced sections and cases where the depth of steel deck is large in comparison to the slab depth, i.e., where the centroid of the deck area does not coincide with the centroid of tensile forces.

2.3.2 - Service load design

2.3.2.1 - Section properties for deflection calculations. In composite sections, the steel deck and concrete act together, with the deck serving as the tensile steel reinforcement for the concrete section subjected to positive bending. For the determination of the flexural properties for deflections, composite moments of inertia are calculated on the basis of assumptions (a) and (d) in the Standard. The steel areas are transformed to equivalent areas of concrete by multiplying by the modular ratio, where the modular ratio is the modulus of elasticity of steel (29,500 ksi) (203 000 MPa) [C.1] divided by the modulus of elasticity of concrete which may be computed as suggested in Section 8.5.1 and Appendix B of the ACI 318 Code [C.32].

At service loads, most slab segments remain uncracked over a significant portion of the depth and length and are consequently stiffer than a fully cracked slab. To assume that the concrete carries no tension is unduly conservative. However, the assumption of a totally uncracked section results in a deflection calculation that is unconservative. Therefore, the Standard states that the average of the cracked and uncracked composite moments of inertia is to be used in calculating predicted deflection. Approximate computations of cracked and uncracked composite moments of inertia are given by the formulas in Appendix B of the Standard. The recommendations for the use of average composite moments of inertia are based on observations from a selected number of test results. In some instances, a more refined composite moment of inertia will be needed.

The Design Standard recommends a simple average for the composite effective moment of inertia, I_d for deflection calculations at service design loads. I_d was

$$I_d = (I_u + I_c)/2 \quad (C.2)$$

based on deflection data from selected specimens used to determine the shear-bond characteristics of composite steel deck sections [C.13].

Additional research at ISU has led to a more involved and potentially more accurate prediction of simply

supported slabs, (C.40-1). This work was for the instantaneous deflection behaviors of noncellular panels or simple roller and pin supports with normal weight concrete were investigated. Examined were deflection data from 142 previously conducted shear-bond strength tests [C.40]. Nine span lengths ranging from six feet (1.83 m) to seventeen feet (5.18 m) were included. Three nominal cold-formed steel-deck depths: 1-1/2" (38 mm), 2" (50 mm) and 3" (76 mm) from six deck manufacturers were used. The overall depth of the composite sections ranged from 3" (76 mm) to 9" (228.6 mm) with widths from 24" (600 mm) to 36" (900 mm). The steel deck thickness ranged from 0.06 in. (1.5 mm) to 0.029 in. (0.74 mm).

A governing instantaneous deflection equation was developed, which predicts the deflections of one-way composite cold-formed steel-deck reinforced concrete slab systems subjected to service design loads [C.41].

Figure 3 in Ref. C.41 shows a typical load-deflection curve of an intermediate span specimen. The figure indicates that the current criteria overestimates the initial stiffness of the section, leading to unconservative estimates of the actual deflection for loads at or below service design load and becomes excessively unconservative as the load approaches ultimate. When comparing the experimental deflection closest to service design load with calculated deflections using Eq. C.2, the current code criteria underestimates the measured deflections by an average of 26 % for all specimens. When considering each span separately, the shorter span, i.e. 5.5 ft. (1.7m) span group deflections were underestimated by about 13% here as for the 16.5 ft. (5m) span specimens the deflections were underestimated by an average of 46 %.

The research method by Porter and Lampert is similar in format with the method currently recommended by ACI 318 for normally reinforced concrete [C.32]. The method uses an effective moment of inertia approach in which the moment of inertia varies between αI_u when the applied moment is below the cracking moment, M_{cr} and approaches the moment of inertia of the steel deck only taken about the composite cracked section neutral axis, I_D , as the load approaches ultimate.

The effective moment of inertia, I_e , is as follows:¹
when

$$M_a < M_{cr} :$$

$$I_e = \alpha I_u \quad (C.3)$$

when

$$M_a \geq M_{cr} :$$

for nominal d_s of 1½" (38mm) and 2" (50mm):

$$I_e = \alpha I_u \left(\frac{M_{cr}}{M_a} \right)^{0.55} + \left(1 - \left(\frac{M_{cr}}{M_a} \right)^{0.55} \right) I_D \leq \alpha I_u \quad (C.4)$$

for nominal d_s of 3" (76mm):

$$I_e = \alpha I_u (M_{cr}/M_a)^{1.3} + (1 - (M_{cr}/M_a)^{1.3}) I_D \leq \alpha I_u \quad (C.5)$$

Results of a linear regression analysis, indicated the stiffness reduction coefficient, k should be obtained using Eq. C.6 - 8 for 1-1/2" (38 mm), 2" (50 mm) and 3" (76 mm) nominal steel deck depths, respectively.

Values of α for 3.4" $\leq h_c \leq$ 5.1": (86 mm $\leq h_c \leq$ 130mm) are as follows:

For a nominal d_s of 1.5" (38mm):

$$\alpha = 1.0 \quad (C.6)$$

For a nominal d_s of 2" (50mm):

$$\alpha = 2.0 - 0.293h_c \leq 1.0 \quad (C.7)$$

For a nominal d_s of 3" (76mm):

$$\alpha = 1.536 - 0.185h_c \leq 1.0 \quad (C.8)$$

The stiffness reduction coefficient, α was necessary because analysis of the load-deflection behavior of composite slabs, indicated 91 % of the specimens had initial stiffnesses below I_u . Though the loads were not theoretically sufficient to cause the moment in the specimen to reach M_{cr} . Equations C.6 - 8 were determined for the ranges of h_c given. The upper limit of $h_c = 5.1$ " (130 mm) should include most slab depths used in normal construction. For values of $h_c > 5.1$ " (130 mm) the authors recommend the value of α be determined using $h_c = 5.1$ " (130mm). For slab depths with $h_c < 3.4$ " (86 mm), the authors suggest that a value of $\alpha = 1.0$ be used [C.41].

2.3.2.2 - Deflection limitations. The deflection limitations adopted for this section are the same as those of Section 9.5 of the ACI 318 Code [C.32], except for the recommended maximum span-to- depth ratios given in Section 1.3.1 of the Standard. Design deflections are computed at service load levels, not for factored loads. The expression for composite moment of inertia used for deflection computations is discussed in Section 2.3.2.1, and approximate equations are given in Appendix B of the Standard.

The deflection due to long-time loading is added to the immediate deflection to obtain the total. For the computation of total deflections, the immediate deflection occurring due to dead load when the concrete is placed can be ignored since this load is sustained by the steel deck. However, the applied load due to shore removal is considered as contributing to long-time deflections. In addition, the instantaneous portion of any deflection of the composite slab due to dead load placed after the concrete has cured but before the elements subject to damage are placed need not be included when calculating the total deflection. However, the long-time effect of this dead load and the slab weight must be considered.

The factor of $\lambda = [2 - 1.2(A_s'/A_s'')] \leq 0.6$ is the same as that in the ACI 318 Code [C.32] except that it may need to be multiplied by a coefficient to account for the effect of the steel deck on reinforcing against the long-time effects of creep and shrinkage or to account for situations where shear-bond slip could cause significant long-time deflections. There may be instances where a shear-bond slip may cause greater long-time deflections under sustained load. Test data

¹ The equations C.3-C.8 were developed in inch-pound units.

are needed for these cases to obtain the appropriate modification coefficients.

The A_s'' term in the above λ factor pertains only to that portion of the steel deck which is in tension under service loads. Thus, a deeper deck section where the composite neutral axis lies within the steel deck section will have $A_s'' < A_s$ with only that portion of deck below the composite neutral axis counted in A_s'' determination based only on a cracked transformed section may be used.

The A_s' term in the above λ factor accounts for the area of steel in compression. This area includes that portion of the steel deck in compression, as well as any other compression reinforcement.

2.3.3 - Special design considerations

2.3.3.1 - Control of shrinkage and temperature effects. The required percentage is less than that specified in the ACI 318 Code [C.32]. This is reasonable because the continuous deck at the bottom of the concrete slab retards evaporation, thereby reducing shrinkage, and also constitutes some transverse restraint. In following the requirements for temperature reinforcement, the designer may eliminate the concrete area that is displaced by the deck rib.

The required area of 0.00075 times the area of concrete above the deck was arrived at by considering the area that 6 x 6 - W1.4 x W1.4 (old designation of 6 x 6 x 10/10) welded wire fabric (WWF) would contribute in a depth of concrete of 3 inches (76 mm) above the deck corrugations computed on a per-foot-of-width basis. In general, for slabs of a total depth of 4 inches (102 mm) or less, the WWF temperature reinforcement may be considered to be located at the center of the concrete above the deck. The designations and standard practices pertaining to WWF are given in Reference [C.35]. The designer should keep in mind that the flexural action of the spans will also help to induce cracks over the supports. Generally, placing the WWF near the top of the slab (over supports) with a cover of 3/4 in. (20 mm) to 1 in. (25 mm) will provide better crack control.

Transverse wires used with some types of decks as shear transfer devices also serve as shrinkage and

temperature reinforcement. The transverse wires must be lapped so as to provide continuity and adequate bond. This is consistent with requirements for welded wire fabric or ordinary reinforcing bars. See the ACI 318 Code [C.32].

2.3.3.2 - Punching shear. Punching shear may be a problem when heavy concentrated loads are applied over small areas. Little testing has been performed to indicate the amount of additional benefit provided by the steel deck. In lieu of such tests, the ACI 318 Code [C.32] procedures are suggested. However, the limiting value for nominal shear stress

of $2\sqrt{f'_c}$ (0.166 $\sqrt{f'_c}$ MPa) used, instead

of $4\sqrt{f'_c}$ (0.33 $\sqrt{f'_c}$ MPa) in view of the lack of complete two-way action. If the designer has a special problem involving punching shear, then performance test data should be obtained.

2.3.3.3 - Two-way action. Floor slabs under heavy concentrated loads such as fork-lift trucks may be subjected to two-way slab action. For such floor slabs, additional reinforcing in the form of welded wire fabric or conventional reinforcing steel will aid in the distribution of the concentrated loads in a direction transverse to the deck corrugations. Transverse wires used as a shear transfer device cannot be utilized as transverse flexural reinforcement unless their effectiveness can be demonstrated by tests.

On the basis of an extensive investigation of 12 ft. (3.7 m) by 16 ft. (4.9 m) floor slabs subjected to four concentrated loads, an effective width concept may be employed for such systems [C.7, C.15]. For shear-bond strength determination, the established effective width is then used as the load-carrying segment of the slab. The effective width depends upon the loaded area, the span, and other factors.

The recommendations in the Standard for the transverse flexural strengths were developed through the research described in Reference [C.7] and summarized in Reference [C.15]. If no supplementary steel exists in the transverse direction, then the transverse moment strengths may be determined by the ordinary flexural formula, $12M_u \leq fS_c$ ($M \leq fS_c$). In this case, the allowable stress, f , is equal to the modulus of rupture of the concrete, f_r . The section modulus of the concrete, S_c , is equal to

$bh^2/6$, where b is the width of the transverse flexural strip and depends upon the controlling mechanism and the rational analysis given in References [C.7] and [C.15]. Considering only the gross section above the deck corrugations is conservative.

When supplementary reinforcing steel is needed to provide added transverse flexural strength, the slab is designed in the transverse direction as an underreinforced slab. This is usually accomplished by neglecting the effect of the steel deck and assuming a conventionally reinforced section above the deck corrugations with the supplementary steel serving as the tensile reinforcement.

Transverse reinforcement to develop two-way action may be needed to provide for discontinuities that exist in a floor slab. Discontinuities due to large header, trench, or utility ducts or holes and to floor outlets placed in the slab should be given careful attention to assure that adequate load-carrying capacity exists around these obstacles. When utility ducts are placed within the slab transverse to the steel deck corrugations, consideration should be given to the effects on the shear-bond and flexural strengths.

2.3.3.4 - Repeated or vibratory loading.

For composite deck systems subjected to a large number of repeated loads the strength of the system is best found by testing for this type of loading. The test specimens should be, as nearly as is practicable, identical to the floor slab involved. In general, an evaluation as recommended in Chapter 3 of the Standard is suggested to obtain the strength associated with the particular mode of failure for a slab subjected to repeated loading. Published [C.42, C.43, and C.44] and proprietary tests previously conducted indicate that many steel deck systems perform quite adequately under fatigue loading with slight reduction in strength.

CHAPTER 3 - PERFORMANCE TESTS

3.1 - Introduction. Performance tests are necessary since each steel deck profile has a shear transfer device with unique characteristics and corresponding unique distribution of horizontal shear forces. The purpose of the tests is to provide data for the ultimate strength relationships contained in Section 2.3.1.5 of the Standard. In particular, a series of tests is to be

performed in order to provide values of ultimate experimental shears for a linear analysis to allow determination of the constants m and k in Eqs. (2-8) and (2-9) in Section 2.3.1.5.1. A complete series of tests for shear-bond determination is required for each steel deck profile. Tests should also be performed to verify, if possible, the flexural mode of failure and behavioral characteristics of the system prior to failure. Information from the various performance tests together with the design equations in Chapter 2 may then be used to obtain tabulated allowable superimposed loads for floor slabs of various span lengths, slab depths, steel thicknesses, surface finishes, and concrete densities and strengths.

In some instances, the Standard provides means of reducing the number of tests from those normally required. Provision is made in Section 3.3 for use of existing test data that are sufficient to establish strength and performance. In addition, Section 3.2 contains provisions to allow a possible reduction in testing requirements by conservatively applying tests from those composite steel deck systems that provide a lower strength in place of testing those of higher strength. The parameters affected include steel thickness, concrete strength, concrete density, and steel surface coating.

Special situations may not be covered in the testing contained in Section 3.2. Section F of the AISI Specification [C.1] is intended to cover the general requirements for evaluation of tests for special cases. Requirements for additional material properties and evaluations not specified are left to the judgment of the test engineer. In general, the intent of the requirements contained in this Standard must be met.

The determination of in-plane diaphragm shear strength requires special tests not covered in the Standard [C.25]. Several modes of failure must be considered [C.26, C.27, C.29, C.30]. Many steel deck manufacturers have proprietary diaphragm composite deck shear values.

The shear transfer device itself may require a special test. For example, in the case of welded transverse wires, the strength of the weld and subsequent shear connector capacity must be determined to establish a proper minimum shear force for these deck sections.

3.2 - Testing of composite slab elements

3.2.1 - Specimen preparation

3.2.1.1 - General. The steel deck and specimen preparation should conform to the general on-site requirements listed in the various sections of the Standard. Within each series of performance tests, the steel deck employed is to have the same profile, surface condition, and thickness. The surface of the steel deck should be free of any foreign contaminants such as grease or oil to ensure proper bonding between the steel and concrete. The surface of the steel deck however, should be in the as-rolled condition. Since rust affects bonding and surface roughness, the surface of the decks for the performance tests must be free of rust. Any change in surface roughness may give erroneous ultimate shear-bond values. Additional discussion of the deck and its surface finish is contained in Section 1.2.1 of this Commentary and in Reference [C.22].

During specimen preparation, care needs to be taken to ensure that the decks achieve uniform bearing on the end reactions. This may require attaching the deck to its support by welds or other means. If any attaching is done, protruding welds or other attaching devices should not exist on the surface of the deck in contact with the concrete. Any alteration in this surface, such as dents or extraneous protrusions, could greatly influence the results of the performance tests and so must be avoided. Studs or other shear connectors used for composite beam action shall not be used in the performance testing to determine the shear-bond design constants unless special tests are being conducted for the purpose of determining the influence of the stud shear connectors or establishing shear-bond values with studs as was done in Reference [C.24].

Placement of the concrete should be in accordance with the standard procedures mentioned in the ACI 318 Code [C.32] and in Sections 1.2.2 of the Standard.

The preparation and design of the performance test specimens also follow the other applicable sections of the Standard. For example, shoring may be required for the longer span test specimens, so that the stresses and deflections do not exceed those specified in Sections 2.2.4 and 2.2.6, respectively. The

simulated shoring and reaction supports should not have any relative movement during casting and curing and shoring supports can not be removed until Section 2.2.3 of the ASCE Standard Practice for the Construction and Inspection of Composite Slabs [C.36] is satisfied.

3.2.1.2 - Dimensions of composite specimens. The lengths and depths of the specimens will be dictated by the characteristics of the particular deck slab system and the need to provide an adequate testing program as prescribed by Section 3.2.3 of the Standard. Longer shear spans usually require longer specimens. Also, the specimens cast for the flexural mode of failure will probably require much longer span lengths than the specimens for shear-bond failure.

Narrow specimens are not desired because of possible width effects resulting from a slight peeling away of the deck along the sides of the specimen. Thus, a lower shear-bond value (per unit width) may result from a 6-inch (152 mm) wide specimen as compared to a 24-inch (610 mm) specimen. A 2-foot (610 mm) width is considered large enough so that any possible edge effects are negligible.

The selection of the width, b_d , of a slab specimen will usually be simply the one repeating steel deck panel width as typically marketed by the deck manufacturer. However, test specimens may also be constructed of multiple deck panel widths so that the longitudinal seam joint (between panels) can be included. Alternatively, with some extra fabrication effort, the typical single panel specimen widths could be fabricated to include the longitudinal seam (by longitudinally cutting the panel sheets). Some manufacturers may feel that the joint contributes to the strength. However, most specimens are anticipated to be constructed with one single deck panel.

3.2.2 - Test procedure. The test procedure consists of loading the test specimens with two concentrated line loads, or with a uniform load, to determine behavioral characteristics, ultimate strength, and the mode of failure. The following failure modes are addressed in the performance section of the Standard:

- (1) shear-bond,

- (2) flexure of an underreinforced section, and
- (3) flexure of an overreinforced section.

Other potential modes, such as end bearing failure or diaphragm slab strength, may result from special situations and will probably be cause for special tests. Some investigations have been conducted on composite steel deck diaphragm strengths [C.19-C.21, C.25-C.27, C.29, C.30]. The primary mode of failure of concern for slabs subjected to gravity loads is that of shear-bond, whereas several other modes are possible for in-plane shear [C.26, C.27, C.29, C.30].

The shear-bond mode of failure for gravity loads is characterized by the formation of a diagonal tension crack in the concrete at or near one of the load points, followed by a loss of bond between the steel deck and the concrete. This results in observable slippage between the steel and concrete at the end of the span. The slippage results in a loss of composite action over the beam segment, referred to as the shear span length, ℓ' . Physically, the shear span is the distance from the support reaction to the concentrated load. Previous tests [C.2-C.18, C.22-C.24, C.28] indicate that the shear-bond mode of failure is the one more likely to occur for most steel deck slabs.

The end slip usually occurs as the ultimate failure load, P_e , is reached and is followed by a significant

drop in loading if hydraulic testing apparatus is used. Some deck systems exhibit small amounts of end slip prior to reaching ultimate load. Figure C3.1 depicts a typical shear-bond failure showing cracking and the associated end slip. End slip normally occurs at only one end of the specimen. Generally, the end slip is less than 0.06 inches (1.5 mm) at ultimate load and is associated with increased deflections and some creep.

The modes of flexural failure for under- or overreinforced deck slabs are similar to those in ordinary reinforced concrete. Failure of an underreinforced deck is primarily characterized by yielding and possible tearing of the entire deck cross section at the location of maximum positive moment. In contrast, failure of an overreinforced deck is primarily characterized by crushing of the concrete at the maximum positive moment section. Small amounts of end slip may be experienced prior to flexural failure. Since flexure of an underreinforced section is usually based on yielding of the steel across the deck section, some yielding of the bottom fibers could occur and still result in a shear-bond mode of failure. In some instances, the top corrugation of the deck may buckle, resulting in a different controlling flexural mode of failure. Care should be taken in interpreting the tests to be certain that the correct mode of failure is being utilized in the respective analysis of the specimen's length.

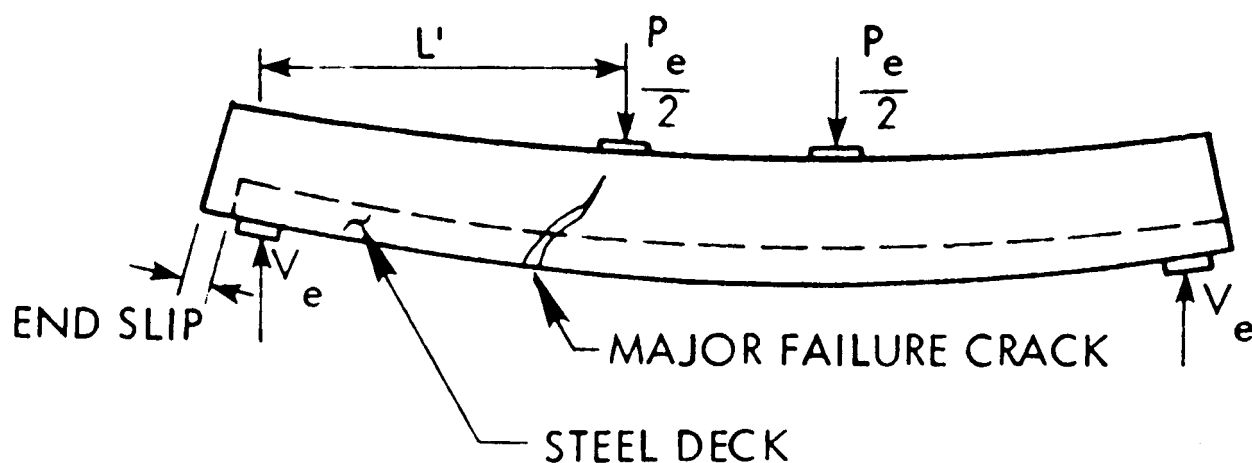


Fig. C3.1. Typical shear-bond failure

3.2.2.1 - Loading of specimens. Loading consists of two symmetrically placed loads, as shown in Figure 3.1, or a uniform load. The two point loads are line loads extending the full width of the specimen. Pressurized air bags, a vacuum system, or a series of dead weights may be used to provide a uniform load over the entire specimen [C.28].

The cushion plates indicated in Detail A of Figure 3.1 may be neoprene bearing pads or other similar material that provides uniform bearing and helps relieve any lateral and longitudinal restraint. The steel plate on top of the neoprene pad provides thickness so that the tips of the wide-flange beam will not bear upon the specimen thereby giving an erroneous shear span.

The load should generally be applied in increments of approximately one-tenth of the estimated failure load. The rate of loading should also receive proper consideration, since rapid loading may unduly affect the results. A small preload to set bearing, load apparatus, and instrumentation may be desirable, after which the loading sequence is commenced from zero load.

Exceptions to testing, as shown in Figure 3.1, would exist for special situations. The possible occurrence of punching shear, for example; would require the engineer to determine suitable standard tests in accordance with recognized procedures.

3.2.2.2 - Instrumentation. Placement of strain gages is suggested at a cross section of maximum moment for those specimens expected to fail in the flexural mode. The strain gages are useful in verifying the assumptions regarding the flexural strength equations contained in Section 2.3.1.5.2 of the Standard and Commentary and in verifying the mode of failure. Placement of strain gages on the top as well as the bottom corrugations of the deck is also recommended to ascertain yielding of the entire steel deck section. Strain gages should be located only on flat portions of the steel deck and not immediately adjacent to or on an embossment or other shear transferring device which might appreciably affect the strain reading.

3.2.2.3 - Recording of data. The careful acquisition of test data is necessary in order to

achieve a valid evaluation of results. The actual measured, as opposed to nominal, dimensions of the composite steel deck slab should be recorded. Appendix C delimitates deck measurements. Where possible, the slab depth, h , should be recorded prior to testing instead of after a shear-bond failure, because during failure the deck is usually displaced downward from the concrete leaving a small void between steel and concrete. Since the exact location of the failure crack is not known beforehand and some variation in the slab depth usually occurs along the length of the specimen, h should be measured at several locations in the areas where the failure is likely to occur. In this way, the h at the actual crack can be found easily and with sufficient accuracy. Measurement of d and t is more easily made prior to construction of the specimens.

Recording of the following information is recommended but not mandatory:

- (1) width of largest crack at approximate service load,
- (2) number of cracks observable at approximate service load, and
- (3) location of failure crack.

In situations where the deck is not continuously shored, increased thickness results from deflections of the deck during casting. See Section 2.2.1 of this Commentary. Since the thickness of the specimens varies along the length and width, measurements should be made at intervals along the length and width. This will also facilitate the determination of h . The depth should be measured at interior points as well as along the edges of the specimen because of the slightly greater depths at the edges.

The determination of the material properties should be based upon a minimum of three concrete cylinders and three steel coupons for each identical set of composite steel deck slabs tested. Tests for material properties should be made in accordance with the following ASTM Standards [C.31]:

- C.39 Standard Test Methods for Compressive Strength of Cylindrical Concrete Specimens
- A370 Standard Methods and Definitions for Mechanical Testing of Steel Products.

The data from the concrete test cylinders is used to determine f'_c , which is the actual strength of the concrete in the slab specimen at the time the test is conducted.

The age of composite slabs at testing may significantly affect the results. Due consideration must be given to the effects of curing and to ensuring that the concrete has properly cured at the interface between the steel deck and the concrete. Slabs utilizing lightweight concrete may require a longer curing time. The actual time of testing is at the discretion of the test engineer, but a minimum time of 28 days is suggested.

Accurate documentation of the behavioral characteristics of the composite steel deck slab during testing is very important in order to identify the mode of failure properly. See Section 3.2.2 of the Commentary for a description of the shear-bond failure mode.

3.2.3 - Scope of tests

3.2.3.1 - Shear-bond tests. Since the internal force distribution between the steel deck and the concrete is quite complex, a semiempirical approach is necessary to establish a uniform method for shear-bond strength prediction. This approach involves a plot of the pertinent parameters known to affect the shear-bond mode of failure. An analysis is made to determine the linear shear-bond plot that best fits the data in order to establish values of m and k for Eqs. (2-8) and (2-9). This procedure is used only for the shear-bond mode.

In order to assure a useful and representative shear-bond plot, the specimens tested must cover a wide range of the parameters. Thus, the Standard requires that tests be performed so that low and high values of the parameters

$$V_u/bd\sqrt{f'_c} \quad \text{and} \quad \rho d/\ell'_i\sqrt{f'_c}, \quad \text{as shown in}$$

Figure 3.2, are obtained. These values will define two extreme regions along the regression line.

Since the major variables (for any given test series) are the depth, h , and the shear span, ℓ'_i , a combination of changes in these two variables usually produces the desired spread for the test data. Recommended values of h and ℓ'_i are shown in Table

3.1. The Regions A and B may be visualized as being at the left- and right-hand extremities, respectively, of the plot in Figure 3.2. The shear span for Region A should be as long as practical and still yield a shear-bond type of failure. For the other extreme, Region B should have a shear span as short as possible, i.e. about 18 inches (460 mm). A minimum of at least 18 inches (460 mm) avoids the effects of having the load too close to the support.

The behavior of slabs with parameters corresponding to Region A tends toward more flexural action, whereas those slabs corresponding to Region B exhibit more shear action. The two regions are intended to bracket the practical range of shear-bond failures.

In order to define a boundary between flexural and shear-bond modes of failure in the test series, the Standard requires that $P_u\ell'_i/2$ be less than 90% of the predicted or expected flexural moment capacity. Because the limiting value of ℓ'_i is not always predictable before testing, additional tests may be necessary to establish that a flexural failure has not occurred. Generally, the use of strain gages will help to ascertain the failure mode.

The tests outlined in the Standard under this section apply to each deck profile. Slabs with the steel deck section oriented in an inverted position will yield different values of m and k and thus require separate testing. Combinations of cellular and noncellular types of decks (see Figure C.3) should not be employed in the same specimen. These two types are normally considered as separate deck profiles. However, the manufacturer may choose to ignore any benefit from the bottom plate of a cellular deck, in which case the results of the noncellular deck tests may be used to determine catalog values for the design of the corresponding cellular deck provided that the cell profile in contact with the concrete is identical for the two deck types. A manufacturer may select from various surface coating options for the deck. Research has shown that shear-bond values for a given deck vary with different coatings [C.22]. The results of the performance tests are, therefore evaluated for the particular deck coating used in the series. Consequently, a complete series of tests should be performed for each surface coating. A particular coating demonstrating a lower strength may be used in lieu of conducting a complete test series on one of higher strength by performing a

verification of at least two (preferably more) direct comparison tests. Thus, the manufacturer may elect to use the results from a coating that gives a lower strength value in lieu of performing the additional tests.

The practice of casting specimens in such a way that two tests can be performed on the same specimen is not recommended. This is because of the unknown effects of damage or permanent deformation that may exist in the unfailed portion of the specimen at the time of testing.

The diagonal tensile strength of lightweight concrete is usually lower than that of normal weight concrete of the same compressive strength [C.32]; hence, the shear-bond strength may also be lower. Therefore, a performance test series is recommended for each concrete type; however, a manufacturer may elect to use the test results from lightweight concrete for both types, provided that the shear-bond values are verified as being lower for the lightweight concrete.

Item (i) of Standard Section 3.2.3.1 is intended to account for reasonable changes of concrete strengths from those tested. If the concrete strength is higher (but within the 1500 psi (10 MPa) limitation) than that specified for design or tests, Standard Section 2.3.1.5.1 can be used to obtain new shear-bond strengths for the new concrete strength in the $\sqrt{f'_c}$ term. Generally, the use of concrete strengths lower than those tested should be discouraged; however, Eq. (3-1) is intended to adjust experimental shears downward to correct for reasonable variations of concrete strengths. Equation (3-1) can be used only if all remaining parameters such as ℓ'_1 , b , d , A_s , m , and k are kept constant.

Examples of ways to arrive at a particular number of needed tests are given in Appendix F of this Commentary. The exact number of tests will vary depending upon the manufacturer's variables and the desirability of obtaining additional information.

3.2.3.2 - Flexural tests. For most composite steel deck slabs, Eq. (2-13) of Standard Section 2.2.1.5.2 adequately predicts the nominal

moment strength. Extensive experience with slabs having decks up to 3 inches (76 mm) in depth provides evidence of consistent performance.

Equations (2-13) and (2-14) may not be valid for slabs having deeper decks because of the assumptions used in their derivation. Likewise these equations may not apply for slabs constructed with steel decks having low ductility. See Section 2.2.1.5 of the Commentary for further discussion.

3.2.4 - Test result evaluation

3.2.4.1 - General. The evaluation of test results primarily serves to obtain the coefficients m and k for determination of the shear-bond strength. The evaluation of the results to determine design equation coefficients for flexural strength is not generally required since this failure mode is similar to that of ordinary reinforced concrete principles.

3.2.4.2 - Shear-bond. The evaluation of the shear-bond strength of a given series of composite steel deck slabs involves establishing a linear relationship as shown in Figure 3.2. When test results are plotted for only two abscissa values of $\rho d/\ell'_1 \sqrt{f'_c}$, the relationship is found by simply joining the centroids of the data groups in Regions A and B with a straight line. However, if test results with more than two abscissa values are used, a linear regression analysis shown in Appendix E of this Commentary will prove useful in determining the shear-bond line.

The Standard requires that the values of the constants m_1 and k_1 are to be reduced for use in Eqs. (2-8) and (2-9). Figure 3.2 shows the resulting "reduced" shear-bond line defined by m and k . The purpose of this reduction is to ensure that the line approaches a lower bound for the experimental values and therefore is somewhat conservative to account for minor laboratory variations in the test results. The magnitude of the reduction is either 15% or 10%, depending on the number of tests used to determine m_1 and k_1 . Naturally, greater confidence is associated with a larger number of tests, and thus a smaller reduction can be used.

The equation obtained for the shear-bond evaluation from Figure 3.2 is as follows:

$$\frac{V_e}{bd\sqrt{f'_c}} = \frac{mpd}{l'_i\sqrt{f'_c}} + k \quad (C.9)$$

or

$$V_e = bd \left[\frac{mpd}{l'_i} + k\sqrt{f'_c} \right] \quad (C.10)$$

where m and k are the reduced values as shown. In the above equations, b is taken as 12 if V_e is in units of pounds per foot of width.

A distinction is made between V_e , the experimentally obtained shear strength, and V_u , the computed or predicted shear strength for use in design. The following equation shows their relationship:

$$V_u = \phi \left[V_e + \frac{\gamma W_s l_f}{2} \right] \quad (C.11)$$

By substituting the value of V_e from Eq. (C.10) into Eq. (C.11) and letting $b = 12$ inches (to place the equation on a per foot basis) and $f'_c = f'_a$, Eq. (2-8) of the Standard is obtained. The strength reduction factor, ϕ , and the term $\gamma W_s l_f / 2$ are discussed in Sections 2.3.1.3 and 2.3.1.5, respectively, of this Commentary; γ values are given in the Standard in Table 2.1.

The test engineer may choose to correct the experimental shear, V_e , obtained from the performance tests by the amount of dead load acting on the composite section. Thus, V_e is replaced with $V_e + \gamma W_s l_f / 2$ in the determination of m and k . If this correction is made, Eqs. (2-8) and (2-9) should not include the $\gamma W_s l_f / 2$ term since it would be reflected in the m and k values so determined. The deck manufacturer's published design tables must reflect this approach if it is used.

The advantages of including $\gamma W_s l_f / 2$ are that all specimens are compared on the same basis and that slightly better results are sometimes obtained in data correlation. However, because the shoring correction is usually small, not much advantage is gained by this technique. Usually, insufficient test results exist on

both shored and unshored specimens to justify an advantage without prejudicing the data toward either Region A or Region B, since shored specimens generally would exist only on the long specimens.

If test results are plotted for more than two abscissa values and the produce the apparently significant

nonlinear relationship between $\frac{V_e}{bd\sqrt{f'_c}}$ and $\frac{\rho d}{l'_i\sqrt{f'_c}}$,

then one of the following courses of action is suggested:

- (1) Review the mode of failure to ascertain if some specimens failed in a mode other than that characteristic of shear-bond, eliminate the extraneous specimens, perform additional tests if necessary, and then once again perform the shear-bond evaluation.
- (2) Use an alternate rational means of analyzing the shear-bond strength.

Other rational methods for analyzing the shear-bond effects may be used if the pertinent parameters contributing to a shear-bond failure (including deck cross section; steel thickness; shear span; concrete weight, strength, and type; shear transfer devices; and method of loading) are considered. These methods could include nonlinear relationships between the parameters. Sufficient tests should be made to establish the method-test variability and substantiate the selection of the appropriate ϕ factors for producing margins of safety comparable to those defined in the Standard.

Depending upon its location in the section, the use of welded wire fabric (WWF) (or other supplementary reinforcing) may be expected to influence the shear-bond strength somewhat. A limited number of slab tests have shown about an 11% increase in shear-bond strength for WWF placed directly on the steel deck [C.17]. Less influence probably would occur if the WWF were placed higher in the section where it would be less effective in resisting positive bending stresses. The deck manufacturer may choose to perform the tests outlined in Standard Section 3.2 to include a minimum WWF, but care should be taken to ensure that the placement is in accordance with the manufacturer's published recommendation. Usually, this WWF placement is one inch (25 mm) from the top of the slab section. Test slab specimens, designed to yield m and k values, can be

constructed to include the WWF reinforcing or, conservatively, omit this supplementary reinforcing. If used, the WWF should be placed at the depth dictated by field practice. Designations of WWF and particular requirements for WWF are contained in Reference [C.35].

3.2.4.3 - Flexure. In situations where the failure mode of a given composite steel deck slab system is flexural, the use of Eqs. (2-13) and (2-14) or the general strain analysis given in Commentary Section 2.3.1.5.2. is required. Generally, the mode of failure can only be established through laboratory testing. Although flexural failures can occur with specimens that exhibit a small amount of end slip, this behavior would tend to invalidate the use of Eqs. (2-13) and (2-14) and the general strain analysis (for slip in the region of maximum moment).

If tests are made to determine the flexural capacity in a direction transverse to the deck corrugations, the analysis discussed in Section 2.3.3.3 of the Standard and this Commentary is the one to be verified in the test evaluation.

For some deck profiles, a flexural failure may not be practical to achieve. If this is the case, the span lengths chosen for design should be limited to the largest tested.

3.3 - Existing tests. Existing test data for one or more representative deck profiles may be used provided that there is an adequate range of values upon which to base the shear-bond line. These values should encompass those composite steel deck slabs to be used in the actual structure. Care must be taken that other properties, such as steel strength and concrete properties, are not appreciably different from the corresponding values to be used in design. The manufacturing tolerances (Standard Section 1.4 and Standard Practice Section 1.3) for the steel deck used in previous testing must also be adhered to.

3.4 - Performance tests. This section serves as providing a means of determining an acceptance of a composite slab system which is already in place (in-situ).

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(Note: Some of the ASTM standards may have SI (metric) unit editions available. If the Standard is used to design in SI units, the user shall investigate whether an SI unit edition of an ASTM standard is available.)

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ASTM A185-90a Standard Specification for Steel Welded Wire Fabric, Plain, for Concrete Reinforcement

ASTM A370-89 Standard Methods and Definitions for Mechanical Testing Steel Products

ASTM A446-89 Standard Specifications for Steel Sheet, Zinc-coated (Galvanized) by the Hot-dip Process, Structural (Physical) Quality

ASTM A496-85 and 496M Standard

Specifications for Deformed Steel for Concrete Reinforcement

ASTM A497-90a Standard Specification for Steel Welded Wire Fabric, Deformed, for Concrete Reinforcement
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APPENDIX A - SI UNITS

Table A.1 - SI Conversion Factors

	TO CHANGE	MULTIPLY BY
LENGTH	in to mm ft to mm ft to m	25.4 (exact) 304.8 (exact) 0.3048 (exact)
AREA	in ² to mm ² ft ² to m ²	645.16 (exact) 0.092903
MASS	lb to kg 2000 lb to 1000 kg lb/ft to kg/m lb/ft ³ to kg/m ³ lb/yd ³ to kg/m ³	0.453592 0.907185 1.48816 16.0185 0.593276
FORCE	lb to N kip to kN lb/in to N/m lb/ft to N/m kip/ft to kN/m psf to kN/m ²	4.44822 4.44822 175.127 14.5939 14.5939 47.880
PRESSURE	lb/in ² to kPa lb/ft ² to kPa kip/in ² to MPa	6.89476 0.04788 6.89476
SECTION MODULUS	in ³ to mm ³ in ³ /ft to mm ³ /m	16387.1 53763.5
MOMENT OF INERTIA	in ⁴ to mm ⁴ in ⁴ /ft to mm ⁴ /m	416231 1365587

APPENDIX B - SECTION PROPERTIES FOR CALCULATING DEFLECTIONS OF COMPOSITE SLABS

The equations given in Appendix B of the Standard are based on standard transformed reinforced concrete section concepts and make use of the assumptions stated in Section 2.3.2.1 of the Standard. These equations are for the composite section subjected to positive bending moment.

The development of Eqs. (B-1) and (B-2) utilizes the standard cracked section concept where the concrete is assumed to carry no tension.

The term nI_{rf} in Eqs. (B-2) and (B-4) represents the transformed moment of inertia of the steel deck about its own center of gravity. This term is negligible for ordinary reinforced concrete, but in the case of a steel deck slab the significance of the nI_{rf} term depends on the relative stiffness of the steel and concrete sections.

The moment of inertia of the steel deck, I_{rf} , is usually taken as the full moment of inertia without reduction for local buckling when considering the composite section. This is because the surrounding concrete aids in preventing the buckling of the compression plate elements.

Equations (B-3) and (B-4) are based upon the standard uncracked transformed section concepts. For these equations the entire concrete area is assumed to contribute to the moment of inertia along with the steel deck. These equations are approximate, based on an idealized shape shown in Figure B.1. The designer may wish to utilize a more exact shape in lieu of Eqs. (B-3) and (B-4).

As discussed in Section 2.3.2.1.(d) of the Standard, the average of the composite cracked and uncracked moments of inertia (as the effective inertia) is recommended for the computation of the composite steel deck slab deflections required by Section 2.3.2.2 of the Standard. This moment of inertia is given by Eq. (B-5). This recommendation is based on selected test results. The test engineer may, however, wish to verify this effective moment of inertia for determining the predicted deflection. That is, a computation of the deflection obtained by using the average of the cracked and uncracked moments of

inertia may be compared with the measured test values. In some cases, a more precise rational analysis of the effective moment of inertia may provide a better prediction of deflections of the service load for Section 2.3.2.2. A discussion of an alternative method is given in Section 2.3.2.1 of this Commentary.

APPENDIX C - DECK MEASUREMENTS

C.1 - General. Appendix C of the Standard provides the necessary measurements needed to know that the steel decking furnished to the job will correspond to the deck that was tested in conjunction with Chapter 3 of the Standard. Of particular importance are the sizes of the shear transferring devices and the thickness of the steel. Measurement of the sheet as furnished is required rather than reference to roll tooling drawings because of variability in the manufacturing process (thickness and ductility).

APPENDIX D - AN ALTERNATE METHOD FOR CALCULATING FLEXURAL CAPACITY OF COMPOSITE SLABS

D.1 - Calculated Flexural Capacities. Composite steel deck slabs have their tension reinforcement principally on the lower surface rather than having the steel encased in concrete as with reinforcing in conventional slabs. Thus, a bending model may be set to identify a reference maximum bending moment but that value is limited by the ability of the deck to transfer the developed tension force to the concrete. This transfer problem is analogous to the bond stress or anchorage problem for reinforcing bars where forces must be transferred along the embedded length. The embossing patterns in bars are standardized and the bars must be totally encased in concrete. With deck reinforcement, however, embossment patterns are non-standard, the reinforcement is not encased, and bond transfer along a shear span is complex. This bending model was developed in work done at West Virginia University under Luttrell's supervision and is summarized in Ref. C.45.

This method aids the engineer in designing a new deck profile by providing a method for selecting an embossment type, for fixing embossment orientation, and for optimization of the embossment spacing. The

equations in this appendix allow the designer to predict the composite performance, prior to entering a large scale test program, and to verify the design through limited testing. The equations given in Appendix D were developed in inch-pound units.

The confirmation test program involves using two steel deck units per slab specimen with the steel deck units attached to supporting steel beams using typical connections. The two unit slab width minimizes the edge flute effect in which the edge-most webs tend to debond earlier than do those nearer the slab center. Insitu testing or the use of end support plates allows panel end support conditions with relatively uniform bearing as expected in field construction. The support beams or plates are not braced apart and thus provide essentially simple supports with the steel panel contacting the support beams in a field-like condition.

D.2 - Flexural Model. The bending strength model establishes a reference flexural capacity M_{et} based on linear strain relationships and limited by the first yield stresses, f_y , on the lower surface. Whether these yield stresses actually develop depends on shear transfer efficiency along the shear span as indicated by K . The location of a neutral surface at y_{∞} is determined as well as the neutral surface y_b for the full steel section. The use of these dimensions leads to a stress distribution over the section and a theoretical bending capacity, M_{et} , which subsequently is modified to $M_t = KM_{et}$ where K accounts for the effective bond force, or anchorage development, over the shear span ℓ'_i . $K = K_3 / (K_1 + K_2)$ with K_3 measuring the influence of the number of cell widths, K_1 the effect of steel deck depth, and K_2 the influence of embossments in the webs. When the bond factor $K \geq 1.0$, the steel deck experiences yield stresses which may extend above the lower surface. The flexural failure is initiated by yielding in the steel; this may be followed by the loss of bond at the interface. When $K < 1.0$, bond failure occurs at steel stresses lower than yield.

D.2.1 - Shear Force Transfer. Shear transfer along ℓ'_i is dependent on the lateral stiffnesses of steel elements in contact with the concrete. With deeper decks, for example, web elements are more flexible with larger D_w/t ratios and de-bond easier than do those with lower D_w/t ratios. Embossment Type I leads to better composite action when the

web-width to embossment-height ratio, D_w/p_h , is below about 35 and Type II embossments perform somewhat better with higher ratios. Embossment profile can have a marked effect on mechanical bond and squarish profiles would perform better than rounded ones. Embossments are stretched-in during manufacturing and tend to be rounded in cross section; the sharper or "squared-up" embossments may be impractical to form during deck rolling operations. For Type I decks, the embossment the orientation should be reversed on facing webs.

The number of cells or flutes in a test specimen influence the bond behavior. Those web elements nearer the slab center are cross connected by the lower deck flat unit whereas the edge-most webs are not so connected. Those webs nearer the edge tend to curl and lose their bonding easier than those that are cross connected. Thus Equation D.9 presents a measure of bonding efficiency as developed from tests involving from 2 to 10 cell widths, C_s , in the test specimens. K_3 is based both in dimensional analyses and the experimental program undertaken at West Virginia University (C.45). Certain wide slab tests have suggested values higher than 1.4 but the majority of tests indicate a conservative upper limit at 1.4.

The constants K_1 and K_2 describe bond efficiency as functions of the steel deck shape and embossment patterns. Larger values imply diminished shear transfer along the shear span. They were determined experimentally from some 180 slab tests involving diverse depths, surface conditions, embossment patterns, and shear spans.

The expression for M_{et} increases with increases in steel deck thickness. If bond anchorage were perfect, the increases would be substantially linear with thickness for a given concrete type. Of the K value terms, K_2 is the least obvious. For Type I decks, K_2 increases with shorter spans of ℓ_{tr} resulting in smaller K coefficients. For Type II decks, K_2 increases nonlinearly with t^2 while M_{et} increases with t . As is observed sometimes in testing, increases of steel panel thickness alone will not necessarily lead to increased measured moments of resistance.

K_2 for Type II steel deck is not particularly sensitive to the specific embossment pattern but does depend on the embossment depth which relates to interlocking the two components. It does depend

moderately on the concrete strength and steel deck thickness. A 60% increase in concrete strength may lead to 10% increase in K_2 and a two-fold increase in thickness may lead to a 50% increase in K_2 . Thus, while M_{et} increases substantially linearly with t , $K M_{et}$ does not.

D.2.3 - Limitations. The application of Appendix D formulas is limited to normal and lightweight structural concrete slabs with steel deck having galvanized surfaces in contact with the concrete. Testing may be done with deck coatings other than galvanized provided this method is used to evaluate web embossments only. Further the formulas are limited to those types of systems tested which involved cell widths C_c up to 12 inches (305 mm), web angles between 55° and 90°, deck depths of 3 inches (76 mm) or less, and webs without reentrant bends. The use of normal or lightweight structural concretes, of similar strengths, has little effect on the slab flexural capacity except through changes in dead loads.

The steel deck cross sections are composed of elements at the deck top, web, and bottom. These elements may be considered as straight elements, meeting at their theoretical intersections, without an appreciable effect on the M_t equation.

D.3 - Confirmation Tests. The confirmation procedure is based on insitu testing for the particular steel deck shape in question. For cases where the units must be moved for testing, substantially the same end conditions will result when the panel ends are secured on flat plates prior to casting the concrete. Fabrication and testing in the same location eliminates problems attendant to moving slabs to test frames that may have end conditions differing from those of the casting location. Further, the steel deck units are to be attached to the support beams with end connections typical of those used in construction assuring close contact between the lower slab elements and the supports. This eliminates uneven end bearing and premature degradation of adhesive bond over steel deck end supports. Typical end connections may include welds or mechanical fasteners such as screws. These connectors will protrude slightly into the concrete but, relative to the embossments already present, will develop virtually no additional shear force transfer.

The test slabs are to be loaded either with uniform loads or with line loads near the third-points of the span. If uniform loading is used, the effective shear span is considered as being $12(\ell_{nr}/4)$. Data are to be collected as described in Section 3.2.2.3. of this standard. The test program is expected to involve a minimum of three "double-wide" specimens for a particular deck profile and embossment type. From these, the average M_o/M_t values are determined for the series and set equal to K_o . If the three slabs all exhibit M_o/M_t values within $\pm 10\%$ of the average value, M_t is multiplied by K_o to arrive at M_n . Thus, the average M_o/M_n value becomes unity and leads to a general strength formula for the tested system. If the test scatter is outside the 10% range, either the lowest two tests are used for K_o or a second series of three slabs is tested. With six test slabs, the coefficient K_o is established from four tests disregarding the two with the highest and lowest scatter.

D.4 - Design Equations. The strength formulas lead to nominal values for both flexural capacities and end shear strength across the slab width. Those systems that have KK_o values at unity or above generally exhibit flexural capacities higher than the bending moments that are present when slip first develops. KK_o values below unity indicate that yield stresses will not develop in the steel.

The M_n or V_n values, along with the appropriate ϕ factors from Section 2.3.1.3, are used with either Equation 2-10 or Equation 2-14 to establish the permissible superimposed uniform loads.

APPENDIX E - LINEAR REGRESSION EVALUATION FOR USE WITH STRENGTH TEST DATA

The equations given in this appendix are for use with the composite deck test results. These equations are used in conjunction with the shear-bond regression shown in Figure 3.2. The coordinates (X,Y) of an experimental point on Figure 3.2 are given by:

$$Y = \frac{V_e}{12d\sqrt{f'_c}} \quad \text{with } b = 12 \text{ in. (305 mm)}$$

$$X = \frac{\rho d}{l' \sqrt{f'_c}} = \frac{A_s}{12 l' \sqrt{f'_c}}$$

The slope, m_1 , and the intercept, k_1 , of the regression line in Figure 3.2 are given by:

$$m_1 = \text{slope} = \frac{\eta \sum XY - \sum X \sum Y}{\eta \sum X^2 - (\sum X)^2}$$

$$k_1 = \text{intercept} = \frac{\sum Y \sum X^2 - \sum X \sum Y}{\eta \sum X^2 - (\sum X)^2}$$

The statistical coefficient of correlation, N and the standard error of estimate, S_y of the linear regression are as follows:

$$N = \frac{\eta \sum XY - \sum X \sum Y}{\{[\eta \sum X^2 - (\sum X)^2][\eta \sum Y^2 - (\sum Y)^2]\}^{1/2}}$$

$$S_y = \left(\frac{\sum Y^2 - k_1 \sum Y - m_1 \sum XY}{\eta - 2} \right)^{1/2}$$

APPENDIX F - DETERMINATION OF NUMBER OF TEST SPECIMENS

The following two examples give an indication of the number of test specimens needed to satisfy the shear-bond test requirements in Chapter 3 of the Standard. These examples were conceived out of a hypothetical situation using the plot of data shown in Figure CF.1 and, therefore, should be construed only as examples. The exact number of tests will vary depending upon the desired test parameters, e.g. steel thickness, concrete density, surface coating, deck profiles, shear transfer device variances, steel type, and concrete strengths desired by the manufacturer. The number of additional tests required because of variations in test results [see Section 3.2.4.2.(b)] will also vary.

Example No. 1

The tests indicated in Table CF.1. provide an example of an absolute minimum number of tests for the following parameters: one profile (e.g. embossed deck), one steel thickness, one concrete density, and one surface coating. If no parameter verifications are needed, the absolute minimum number of tests in Table CF.1. becomes only four. The minimal tests in Table CF.1. may need modification for the following reasons:

- (1) The companion tests confirm that the original shear-bond provides the lower-bound strength; i.e., an 18-gage deck produces a higher strength than the tested 20-gage thickness; a lightweight concrete density gives lower strength than normal concrete; and a galvanized coating produces a lower strength than a phosphatized coating.
- (2) Previously conducted tests are applicable if they satisfy the Standard.
- (3) These tests are based on shear-bond failure, since the flexural mode very seldom exists.

Example No. 2

This example offers a practical maximum number of tests for the following parameters: three steel deck profiles, two surface coatings, three steel thicknesses, two concrete densities as shown in Table CF.2.

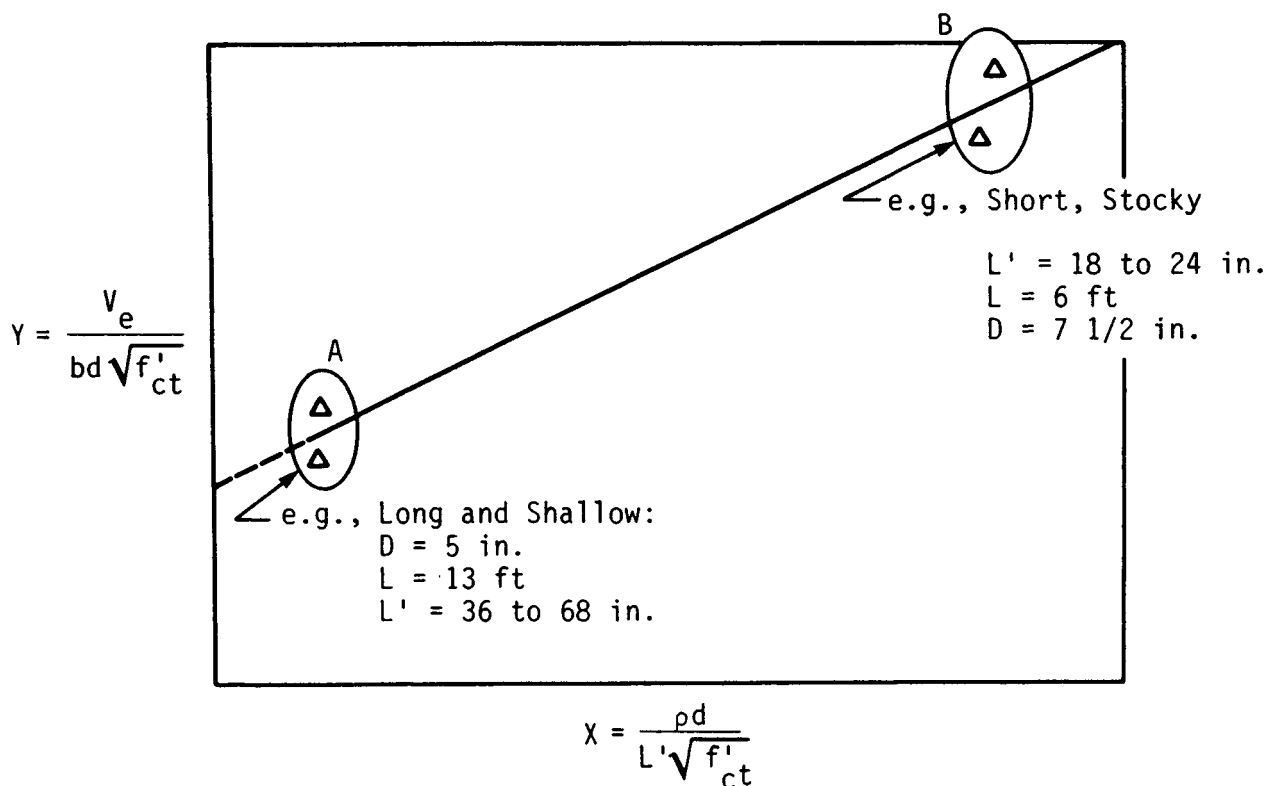


Fig. CF.1. Shear-bond regression example

Table CF.1 - Example of minimal number of tests

Number of Tests	Purpose of Test
4	Shear-Bond Line (provided scatter is within 15%)
2	Companion for Surface Coating (e.g., one short span for Region B and one long span for Region A)
2	Companion for Concrete Density (e.g., one in Region A and one in B)
2	Companion for Thicker Steel (e.g. one in Region A and one in B)

Table CF.2 - Example of number of tests incorporating additional parameters

Number of Tests	Test Parameter Combinations
4	1 1/2" (38 mm) deck, 20 gage, lightweight concrete, galvanized
4	1 1/2" (38 mm) deck, 18 gage, lightweight concrete, galvanized
4	1 1/2" (38 mm) deck, 16 gage, lightweight concrete, galvanized
2	1 1/2" (38 mm) deck, 20 gage, normal concrete, galvanized
2	1 1/2" (38 mm) deck, 20 gage, normal concrete, phosphatized
2	1 1/2" (38 mm) deck, 20 gage, lightweight concrete, phosphatized
18	— Total (provided minimum scatter and lower-bound assumptions prevail)
18	— Same as above for 2" (50 mm) deck
18	— Same as above for 3" (76 mm) deck
Grand Total	54 for 3 steel deck profiles, 2 surface coatings, 3 steel thicknesses, and 2 concrete densities

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ASCE Standard Practice for the Construction and Inspection of Composite Slabs

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ASCE Standard Practice For Construction And Inspection Of Composite Slabs

CHAPTER 1 - GENERAL

1.1 - Introduction

1.1.1 - Composite slabs. Composite slab construction is a system comprising normal weight or lightweight structural concrete placed permanently over cold-formed steel deck in which the steel deck performs the dual role of acting as a form for the concrete during construction and as positive reinforcement for the slab during service.

1.1.2 - Objective and scope. This Standard Practice, including appendices, presents provisions applicable to composite slabs. This Standard Practice is intended as a guide document and not as a mandatory standard practice for:

- (a) good construction practice, and
- (b) inspection procedures.

This Standard Practice includes the appendix, unless specifically exempted.

For provisions on design and testing, see ASCE 3 Standard for the Structural Design of Composite Slabs [1]¹.

1.1.3 - Inch-pound units. Expressions appearing in this Standard were developed in the Inch-Pound unit system. SI units as given in parentheses and Appendix A of this Standard are for information only and are not part of the Standard.

1.2 - Materials

1.2.1 - Steel deck. Grades of steel used in fabricating steel deck units shall be as described in Section A3 of the AISI Specifications for the Design of Cold-Formed Steel Structural Members [2]. The deck surface coating shall be appropriate to the

environment of the structure and shall be specified by the engineer.

1.2.2 - Concrete. Materials for concrete, including admixtures and reinforcing, shall comply with Chapter 3 of ACI 318 [3]. Concrete shall be proportioned to meet the durability requirements of Chapter 4, and Sections 5.1 through 5.5, and evaluated in accordance with Section 5.6 of ACI 318 [3]. Calcium chloride or admixtures containing chloride from other than impurities from admixture ingredients shall not be used in concrete. Maximum water soluble chloride ion concentrations from all sources in hardened concrete shall not exceed the limitations of Section 4.3 of ACI 318 [3]. See Section 2.2 for requirements on concrete placement.

1.3 - Tolerances and weld sizes

1.3.1 - Delivered minimum thickness. The uncoated minimum thickness of the steel deck as delivered to the job site shall not, at any location, be less than 95% of the design thickness; except at bends, such as corners, due to cold forming effects.

1.3.2 - Tolerances. Reasonable tolerances from specified dimensions for shear transfer devices shall be as follows:

- (a) Horizontal spacing of shear devices
 $\pm 1/4$ in. (6.5 mm)
- (b) Dimensions of shear transfer devices-10%
- (c) Depth of embossments or indentions (for those decks which have this type of shear transfer device) -10%
- (d) For decks utilizing holes as shear transfer devices, the diameter of the holes shall have a tolerance of $\pm 1/16$ in. (± 1.6 mm)

1.3.3 - Minimum weld sizes. The sizes of welds for shear transfer devices shall not be less than those tested in Chapter 3 of ASCE 3 [1].

¹ Numbers in brackets denote REFERENCES listed after Chapter 3

CHAPTER 2 - CONSTRUCTION PRACTICE

2.0 - Notation

f'_c = specified compressive strength of concrete, psi

2.1 - Steel deck

2.1.1 - Damage control. The steel deck shall not be damaged when stored, handled and installed, or when used as a working platform. In the event of damage to the deck or its protective coating, the engineer shall determine whether or not the damaged deck shall be replaced, reinforced, or repaired. The engineer shall consider the extent or nature of the damage, the effect on the structural properties, and the relation of the damaged deck to the specific function and application of the composite slab.

2.1.2 - Connections. Every deck sheet shall be fastened to each support at each side of the sheet and where required, through one interior rib so that the average spacing of fastenings along the supports is not more than 12 in. (305 mm). At the side laps, the sheets shall be fastened to each other to control differential deflections or a separation of sheets prior to and during placement of concrete.

2.1.3 - Welding. All welding shall be done in accordance with AWS D1.3 [4]. If headed shear studs are used, AWS D1.1 [4] shall also apply. A minimum weld electrode or filler strength of 60 ksi (400 MPa) shall be used.

Arc spot welds (puddle welds) to supports shall have a minimum effective diameter (weld nugget) of 1/2 in. (13 mm), or an elongated weld (arc seam weld) with a minimum effective nugget width of 3/8 in. (10 mm) and a minimum length of 1 in. (25 mm). Weld metal shall penetrate all layers of decking and shall be fused to the structural members. Weld washers shall be used for steel decks less than 0.0280 in. (0.7 mm) thick.

2.1.4 - Mechanical fasteners. Mechanical fasteners (powder actuated, pneumatically actuated, screws, bolts and side crimps) are recognized as anchoring methods, provided the spacing of such fasteners satisfies the design criteria. Documentation in the form of test data, design calculations, and/or

design charts shall be submitted by the fastener manufacturer as the basis for obtaining approval by the engineer. Where spacing, as specified by the fastener manufacturer, exceeds 12 in. (305 mm) and trough spacing is equal to or exceeds 12 in. (305 mm), the deck manufacturer shall be consulted by the engineer to specify additional fasteners to stabilize the given profile against sideslip of any unfastened troughs under construction loading.

2.1.5 - Working Platform. When used as a working platform, the steel deck shall be designed and connected to supports to support loads during construction. The steel deck shall be protected by timbers or equivalent material from damage by static or impact loading of structural steel, equipment, or other heavy objects placed on it. An investigation shall be required to determine if shoring is needed to support the working platform.

2.2 - Concrete

2.2.1 - Planking. Planking shall be placed when buggies are utilized in the concreting operation. Planks shall be of adequate stiffness to transfer loads to the steel deck without damaging the deck. Buggies shall operate on planking only.

2.2.2 - Concrete placement. Prior to concrete placement, the steel deck shall be free of soil, debris, oil, standing water, loose mill scale or coating, and all other foreign matter. Concrete shall be placed such that the deck will not be damaged by impact or overloading. Wet concrete shall be placed in a uniform manner over the supporting beams and spread towards the center of the deck span. Construction joints shall be located over structural supports, or other locations approved by the engineer, in such a manner as to maintain composite action of slab area adjacent to joint. Where present, joints along composite beams shall be constructed as specified by the engineer.

Concrete shall be cured until the required concrete strengths are attained and to inhibit surface cracking. Moisture retention measures shall be continued for at least 7 days after concrete placement.

Concrete shall be conveyed, placed and cured in accordance with Sections 5.9 through 5.11 of ACI 318 [3]. Concreting during cold or hot weather shall

comply with Section 5.12 or 5.13, respectively, of ACI 318 [3].

2.2.3 - Shore removal. Temporary shore supports shall not be removed until a concrete strength of $0.75 f'_c$ has been attained or as directed by the engineer.

2.3 - Holes and hole reinforcement

2.3.1 - General. The structural framing and/or reinforcement for all openings shall be in accordance with the engineer's design as shown on the contract drawings.

2.3.2 - Small openings. Small openings as defined by the engineer on the job drawings shall be formed, or formed and reinforced, by the construction trade requiring the openings and approved by the engineer.

2.3.3 - Slab openings. Slab openings located on the drawings shall be structurally framed or formed and reinforced by the contractor. For openings that are formed (concrete excluded from formed area), the

deck shall be cut after the concrete attains the specified strength.

2.3.4 - Coring of holes. Any coring of holes shall be approved by the engineer.

2.4 - Conduits in composite slab

Conduits are permitted in deck slabs subject to local code requirements and fire rating considerations. When conduit is installed in the slab, it shall be limited to conduits having a diameter of 1" (25 mm) or less, or less than 1/3 the concrete cover, and no crossovers occur, and conduit is spaced at least 18" (450 mm) apart with a 3/4" (20 mm) minimum cover.

CHAPTER 3 - INSPECTION REQUIREMENTS

An inspector approved by the authority having jurisdiction shall verify that the material delivered to the job site is in accordance with the contract documents; and that the installation of the steel deck, and the subsequent placement of the concrete, is in accordance with the provisions of Chapter 2. Refer to Appendix B if specific dimensions are needed.

APPLICABLE DOCUMENTS

1. *ASCE 3-91, Standard for the Structural Design of Composite Slabs*, American Society of Civil Engineers, Steel Deck with Concrete Standards Committee of Management Group F, Codes and Standards, 345 East 47th Street, New York, New York, 10017-2398.
2. *Specification for the Design of Cold-Formed Steel Structural Members*, American Iron and Steel Institute, 1101-17th Street N.W., Washington, D.C., 20036, August 19, 1986, with December 11, 1989 Addendum.
3. *American Concrete Institute Standards*, P.O. Box 19150, Redford Station, Detroit, Michigan, 48219.
- ACI 318-89, Building Code Requirements for Reinforced Concrete*, November, 1989
or
ACI 318M-89, Building Code Requirements for Reinforced Concrete, (Metric-SI), 1989
4. *American Welding Society Standards*, 550 N.W. 42nd Avenue, Miami, Florida, 33126.

AWS D1.1-90, Structural Welding Code - Steel, 1990
AWS D1.3-81, Specifications for Welding Sheet Steel and Structures, 1981

APPENDIX A - SI UNITS

Table A.1 - SI conversion factors

	TO CHANGE	MULTIPLY BY
LENGTH	in to mm ft to mm ft to m	25.4 (exact) 304.8 (exact) 0.3048 (exact)
AREA	in ² to mm ² ft ² to m ²	645.16 (exact) 0.092903
MASS	lb to kg 2000 lb to 1000 kg lb/ft to kg/m lb/ft ³ to kg/m ³ lb/yd ³ to kg/m ³	0.453592 0.907185 1.48816 16.0185 0.593276
FORCE	lb to N kip to kN lb/in to N/m lb/ft to N/m kip/ft to kN/m psf to kN/m ²	4.44822 4.44822 175.127 14.5939 14.5939 47.880
PRESSURE	lb/in ² to kPa lb/ft ² to kPa kip/in ² to MPa	6.89476 0.04788 6.89476
SECTION MODULUS	in ³ to mm ³ in ³ /ft to mm ³ /m	16387.1 53763.5
MOMENT OF INERTIA	in ⁴ to mm ⁴ in ⁴ /ft to mm ⁴ /m	416231 1365587

APPENDIX B - DECK MEASUREMENTS

B.0 - Notation

B_c	=	repeating width of one cell, in.
d_d	=	overall depth of steel deck profile, in.
D_w	=	length of web segment of profile cell, in.
N_b	=	length of embossment along its base, in.
N_t	=	length of embossment along its top ridge, in.
$n_1, n_2, \text{etc.}$	=	dimension along each straight segment of curved embossments, in.
p_h	=	embossment depth, in.
q	=	horizontal distance between embossment centerline, in.
w	=	width of embossment, in.
W_b	=	width of profile cell across its base, in.
W_t	=	width of profile cell across its top, in.

B.1 - General

Predicting the superimposed slab capacity based on shear-bond using the equations in the ASCE Standard for the Structural Design of Composite Slabs [1] requires the measurement of the dimensions of the embossment and its orientation on the deck sheet.

B.2 - Measurements

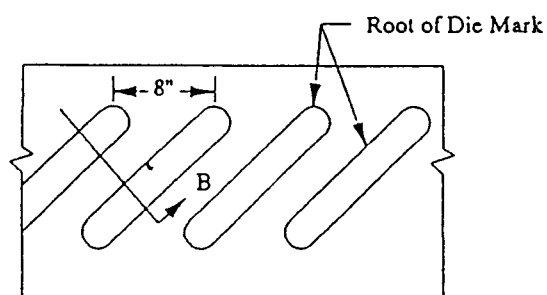
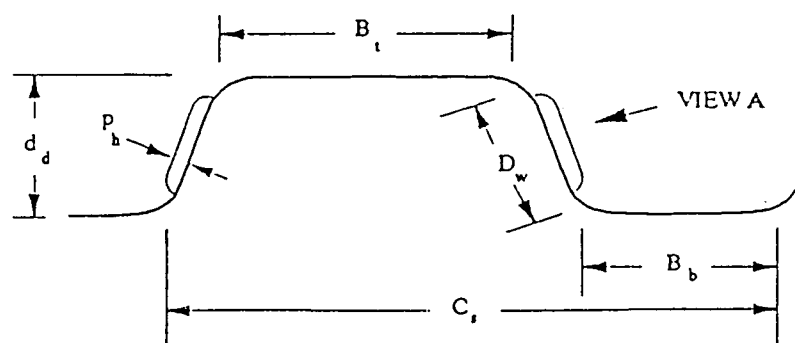
B.2.1 - Embossments. Measurement of

embossments shall be taken equally on all rows of embossments on all elements having embossments. A minimum of two embossments per row per element per test specimen shall be measured. Corresponding measurements of different rows shall be averaged.

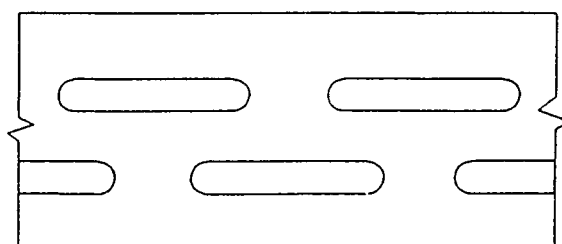
B.2.2 - Measuring devices. Measurements shall be taken using either a dial gage or a depth micrometer with a tip tapered to 0.015 in. (0.38 mm) to accurately measure depth. A straight edge shall be used as a reference from the crest or ridge of adjacent embossments so that the measurement is taken on the raised embossment side of the sheet. Alternatively, measurement shall be taken directly from the recessed side of the embossment.

B.2.3 - Dimensions to be recorded for straight embossments. For straight embossments, dimensions to be recorded are the embossment depth, (p_h), length of embossment along the base (N_b) and along the ridge (N_t); width along the base (B_b) and width along the ridge (B_t); and the horizontal distance(s) between embossment centerlines. See Figure B.1.

B.2.4 - Dimensions to be recorded for curved embossments. For curved embossments, dimensions to be recorded are the distances along each straight segment of the base of the embossment ($n_1, n_2, \text{etc.}$). These dimensions are added together to calculate N_b and N_t instead of attempting measurement. See Figure B.1.



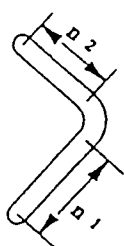
VIEW A
TYPE I



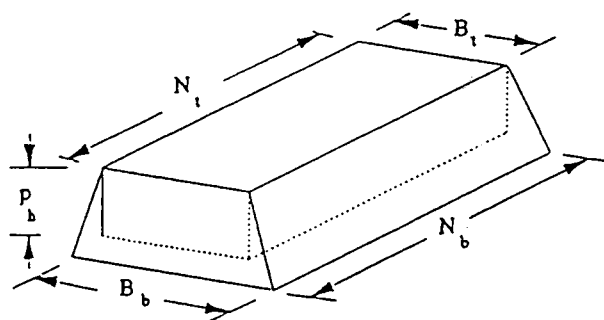
VIEW A
TYPE II (2 Rows)



SECTION B



CURVED EMBOSMENT



IMAGINARY "BOX" OVER EMBOSMENT
AT ROOT

Figure B.1 - Deck measurements

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(This Commentary is not a part of the Standard. It is included for information purposes.)

Commentary on ASCE Standard Practice for Construction and Inspection of Composite Slabs

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(This Commentary is not a part of the Standard. It is included for information purposes.)

Commentary on ASCE Standard Practice for Construction and Inspection of Composite Slabs

CHAPTER 1 - GENERAL

1.0 - Notation.

- A_s = cross-sectional area of steel deck, in.²/ft. of width
 f_c' = specified compressive strength of concrete, psi
 t = thickness of steel deck exclusive of coating, in.

1.1 - Introduction.

1.1.1 - **Composite slabs.** Cold-formed steel deck sections are used in many composite floor slab applications. In such applications, the steel deck serves not only as the form for the concrete during construction but also as the main tensile reinforcement for the bottom fibers of the composite slab. The term "composite slab" is applied to floor systems where the steel deck has some mechanical means of providing positive interlocking between the deck and the concrete. Two examples of composite floor slabs are given in Figures C1.1' and C1.2. The system shown in Figure C1.1 illustrates composite action by means of embossments or indentations in the plate sections of the steel deck. The composite action shown in Figure C1.2 is achieved by means of transverse wires welded to the top horizontal surfaces of the steel deck. In addition, composite beam action may be achieved between the composite steel deck floor slab and the support beams by attaching studs or approved effective shear connectors, as shown in Figure C1.2.

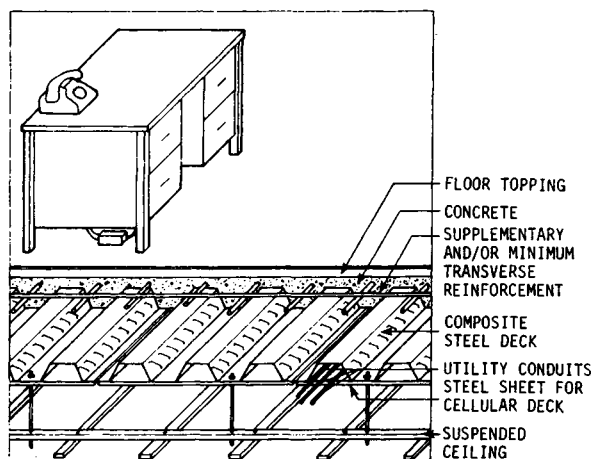


Fig. C1.1 - Typical building floor construction utilizing cold-formed steel decking

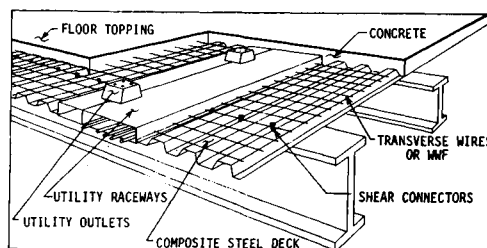


Fig. C1.2 - Typical building floor construction utilizing cold-formed steel deck with composite support beams

¹ References, figures, and equations with numbers beginning with "C" appear in this document. Citations of other section, equation, table, and figure numbers refer to the Standard Practice document; cross references to sections within the Commentary are clearly indicated. Each section number of this Commentary corresponds to the same numbered section of the Standard Practice [C.1].²

² Numbers in brackets denote REFERENCES listed after Chapter 3.

In some instances, the steel deck may be considered only as a form for the concrete, in which case the deck and the concrete are not assumed to act compositely. For this noncomposite application, the deck is designed to carry the construction loads, deck dead load, and wet concrete dead load prior to curing of the concrete. After curing, the deck usually is not counted on for slab reinforcing; rather a conventionally reinforced concrete slab is designed to carry the applied loading.

In addition to acting as a form for the concrete and as the main tensile reinforcement (in positive bending), the steel deck may also (1) serve as a working platform for the workmen, materials, and equipment during construction; (2) provide support for a suspended ceiling as shown in Figure C1.1 or serve as the ceiling surface itself; (3) provide raceways as shown in Figure C1.2 for utility distribution; and (4) provide lateral bracing of steel beam flanges.

1.1.2 - Objective and scope. The guidelines contained in the document **ASCE Standard Practice for Construction and Inspection of Composite Slabs**, [C.1]^{2,3} hereinafter referenced as the Standard Practice are considered *minimum* recommendations. In some cases, construction and job site conditions will dictate more stringent requirements than those in the Standard Practice.

The design method for the loading on the steel deck prior to the curing of the concrete is based on accepted elastic principles. The design criteria for composite steel-deck slabs is contained in the document **ASCE 3 Standard for the Design of Composite Slabs** [C.2] and suggestions, background information, explanations are contained in the associated Commentary document [C.3]. The design considerations for deck properties and allowable stresses under this stage of loading follow the American Iron and Steel Institute (AISI) **Specification for the Design of Cold-Formed Steel Structural Members** and related documents [C.4]. Various standards of the AISI and other organizations are cited in these Specifications and Commentary. Since these standards are revised from time to time, revised editions should be checked to ascertain appropriateness to other applicable codes and this

Standard Practice. In the event of any conflict with other code and specification documents, the design engineer or building code official must determine the applicable governing sections.

The Standard Practice is not intended to cover highway bridge deck construction where the various specifications of the American Association of State and Highway Transportation Officials may apply. This does not imply that composite or noncomposite decks cannot be utilized for such applications.

Reference C.3 contains many citations of background, information and research conducted on steel deck composite slabs. These references, in general, are not repeated in this Standard Practice Commentary, but should be used for additional information, as needed.

1.1.3 - Inch-pound units. This Commentary was developed in the Inch-Pound Unit system; however, SI units are shown in parenthesis and in Appendix A. The SI units shown in parenthesis and in Appendix A of this Commentary are for information only.

1.2 - Materials

1.2.1 - Steel deck. The more common grades of structural quality carbon and high-strength, low-alloy sheet and strip steel used for the manufacturing of cold-formed steel decks are ASTM A446 and A611 [C.5]. Other approved grades of steel are specified in the AISI Specification [C.4]. The minimum specified yield strengths of these steels range from 33 ksi (230 MPa) to 80 ksi (550 MPa).

Each steel deck manufacturer should specify the appropriate standard materials being used. Mechanical properties of the steel deck material are determined in accordance with the requirements of the AISI Specification [C.4] or the pertinent ASTM Standard [C.5].

Manufacturers of steel decks with welded transverse wires should indicate the ASTM Standard designation pertaining to the wires. For example, deformed wire

² Unless otherwise noted, the "Standard or Standard Practice" referred to in this Commentary is the document entitled "Standard Practice for the Construction and Inspection of Composite Slabs" [C.1].

is to conform to ASTM A496, and smooth wire to ASTM A82 [C.5].

Many steel decks are formed from zinc-coated, pretreated, or prepainted sheets (e. g. using iron phosphate or zinc-coating treatment). Zinc coating usually ranges from light G01 up to G90 galvanizing Z01 to Z90 (0.90 oz/ft² (90 g/m²)) as specified by ASTM A525 [C.5] or Federal Specification QQ-S-775d [C.6]. Some applications may require heavier galvanizing. Past applications have included as high as 1.25 oz/ft² (125 g/m²).

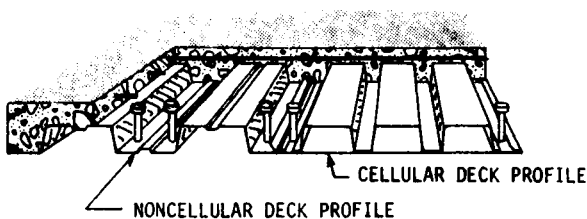


Fig. C1.3 - Illustration of a typical blended cellular and noncellular type of deck profile

Some decks may have a phosphate treatment on the top side and a precoated surface or painted on the bottom side.

If the steel deck is exposed to a corrosive environment, the surface treatment should adequately protect the steel. The type of surface treatment must be specified by the designer. Heavy rusting of the deck should not be allowed to take place. However, light rust (tight rust), such as might occur prior to installation, generally will not affect the structural integrity or performance of the deck. This light rust is acceptable in normal applications where the deck will not be subjected to a corrosive environment or continued rusting after installation. The effects of this tight rust and other coatings were compared and the corresponding strengths and behavioral differences are given in Ref. C.7. See also Section 3.2.1.1 [C.3] and Section 2.2.2 of this Commentary. For additional recommendations on protective coatings refer to the AISI Supplementary Information on the Specification for the Design of Cold-Formed Steel Structural Members [C.4]. In general, the composite deck must be designed to last for the entire life of a structure since it serves as the

primary positive bending moment reinforcement. Thus, a minimum surface finish is usually required. When using a galvanized sheet in a corrosive environment, a minimum G60 galvanizing is suggested. Some applications will require a heavier surface protection. The surface condition of supplementary reinforcing steel must meet the requirements of Sections 3.5 and 7.4 of ACI 318 [C.8].

Minimum thickness of steel deck is usually stipulated in the job Specifications. In general, the thickness, *t*, should not be less than 0.023 inches (0.58 mm).

1.2.2 - Concrete. The concrete for composite slabs must be of the structural quality established by project specifications. ACI standards that may apply in composite slab construction are:

ACI 117	Standard Tolerances for Concrete Construction and Materials [C.8]
ACI 201.2R	Guide to Durable Concrete [C.8]
ACI 211.1	Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete [C.8]
ACI 211.2	Standard Practice for Selecting Proportions for Structural Lightweight Concrete [C.8]
ACI 301	Specifications for Structural Concrete for Buildings [C.8]
ACI 302.1R	Guide for Concrete Floor and Slab Construction [C.8]
ACI 304	Recommended Practice for Measuring, Mixing, Transporting, and Placing Concrete [C.8]
ACI 305R	Hot Weather Concreting [C.8]
ACI 306R	Cold Weather Concreting [C.8]
ACI 308	Standard Practice for Curing Concrete [C.8]
ACI 309	Guide for Consolidation of Concrete [C.8]
ACI 318 or ACI 318M	Building Code Requirements for Reinforced Concrete [C.8]
ACI 318.1 or ACI 318.1M	Building Code Requirements for Structural Plain Concrete [C.8]

Steel deck floor slabs may be constructed with either normal, semi-lightweight or lightweight structural concrete. The minimum compressive strength (*f'_c*) is 2500 psi (17 MPa).

ACI 318 [C.8] gives requirements for maximum size of aggregate in relation to the clearance dimensions and reinforcing bars (in the case of additional steel for negative moment reinforcing, etc.). The size limitations on aggregates are necessary to ensure proper encasement of the reinforcing bars and penetration into steel deck ribs and/or holes. In addition, aggregate gradation and proper concrete mix proportions are important to achieve the desired concrete consistency and strength.

For those sections using holes as shear transfer devices, the maximum aggregate size should be smaller than the hole. This prevents any possible plugging by a larger piece of aggregate restricting the proper flow of the concrete through the hole.

The deck profile itself may restrict the maximum size of the aggregate. For example, a deck section having corrugations close together may have the maximum aggregate size dictated by the spacing or geometry of the corrugations. This limited clearance should not occur in most decks because adequate distance between corrugations must also be provided for welded shear connectors that may be used with supporting composite beams.

Aggregate size limitations may occur in relation to the minimum depth of concrete over the deck (see Section 1.3.2 of the Standard for the Structural Design of Composite Slabs [C.2]). Also see corresponding Commentary [C.3]). Additional aggregate size limitations with regard to proper clearances between reinforcing steel placed over the deck and the top of the deck should be in accordance with accepted standard concrete practices.

Chloride salts are detrimental because the electrolytic action resulting from the presence of salt may cause corrosion of the steel. Thus, the Standard Practice prohibits the use of admixtures containing chloride salts. In addition, the chloride ion content of the mixing water and the aggregate should not be excessive. Section 4.3 of the ACI 318 Code [C.8] gives limitations for chloride ion content. Section 3.4.2 of the 1977 edition of the Commentary (ACI 318) states that a chloride ion content of 400 ppm or 500 ppm might be considered dangerous for applications involving prestressed concrete or aluminum embedments and suggests that levels well below those values be maintained if practical. The ACI 201 Guide to Durable Concrete [C.8] suggests

limits that are slightly more restrictive than ACI 318. Though not strictly applicable to composite steel deck slab construction, these recommendations are included for general guidance.

1.3 - Tolerances and weld sizes

1.3.1 - Delivered minimum thickness. The delivered minimum thickness of steel, t , is based upon the uncoated thickness. The 95% criteria applies to the straight element portions of the deck's cross-section since the cold-forming process can cause variations in the bends. In general, the total net A_s for the whole deck section should not be less than 95% of the specified A_s used for design. The uncoated measurements can be taken by either removing the coating and measuring the resulting thickness or by using an electronic device which does not count the coating. If the coating is removed, care must be given to insure that part of the steel material was not also removed.

1.3.2 - Tolerance. For a commentary discussion of tolerances, see Section 1.4 of the Design Standard Commentary [C.3]. On-site inspection of the deck should take place to insure that it is representative of that tested within tolerance of the Standard Practice, but need not be an exact duplicate. The size and arrangement of the shear-transferring devices should also be essentially the same as those tested and used for design.

The tolerances given indicate the maximum variation allowable between the deck specified and that furnished in practice. Tolerances for the depth of the slab, the location of negative moment reinforcement, and the location and style of supplemental reinforcement not given by the Standard Practice are governed by the standard tolerance guidelines in Sections 7.5 and 7.6 of ACI 318 [C.8].

Additional tolerances beyond those listed in the Standard Practice may be applicable. As a guide, the deck supplied by the manufacturer to the job site should not vary from the tested deck beyond the following:

- (1) Average overall deck depth should not be 1/16" (1.6 mm) less than that specified; but not more than 1/8" (3.2 mm) less in any individual rib.

- (2) The overall width of a deck should not change by more than $\pm 3/8$ (10 mm) to $1/2$ inch (13 mm).
- (3) The cross sectional area of the steel deck should not result in a decrease of more than 5% from the specified section properties.
- (4) The Standard indicates a $\pm 1/4$ inch (6.5 mm) tolerance on the spacing of shear devices. This tolerance should not exceed $\pm 10\%$ of the individual spacing.

1.3.3 - Minimum weld sizes. Some steel deck sections achieve their composite action by having transverse wires or other items welded to the deck. The sizes as well as the strengths of these welds need to be at least as great as those used in the testing of the deck when the design coefficients were determined per chapter 3 of ASCE 3 [C.2].

CHAPTER 2 - CONSTRUCTION PRACTICE

2.1 - Steel deck

2.1.1 - Damage control. Proper installation of the deck is essential for the safety of the construction personnel and for long-term functioning of the steel deck slab. Proper care should be taken during storage and handling to prevent damage to the deck. Bending of the deck from bundling, lifting slings, or other causes should be avoided. Damage to the side laps should be avoided so as not to impair the alignment and seating of the laps. Minor damage to the deck coating will generally not need repair unless the deck is to be architecturally exposed or is subject to a hostile environment. Galvanized coating, because of the superior protection that it provides, can sustain greater localized damage than paint coatings and still perform its function.

Steel decks used as working platforms during the construction phase are generally subject to some distortions or deformations. Strengthening of a damaged deck might be accomplished by the use of shoring or the addition of reinforcing bars in the slab. In some cases a fireproof coating applied to the underside of the deck may be desirable to acquire a higher fire rating. Fire ratings generally have been established by Underwriter's Laboratory [C.9]. In some instances, the thickness of the slab or of the steel deck needed to satisfy the fire rating requirements may take precedence over the normal

load-carrying requirements.

On-site inspection of the deck should take place to ensure that it is substantially the same as that tested within the tolerance of this Standard Practice. The size and arrangement of the shear-transferring devices should also be essentially the same as those tested and used for design.

2.1.2 - Connections. Steel deck may be connected to the supports and the individual sheets (panels) to each other by a variety of means, including welds, screws, bolts, crimping, and button punching. Adequate bearing length at the supports is needed to provide room for good connections. Connections on 12-inch (305 mm) (average) centers along the supports should prevent the deck from slipping off the support beams during construction. Calculations for diaphragm loads or other conditions may dictate a reduction in this spacing. For the purpose of preventing the deck from slipping off the supports, a 2-inch minimum bearing distance should suffice; however, additional bearing lengths may be controlled by design loads on the deck during construction or on the composite section for full live and dead load applications.

2.1.3 - Welding. Steel decks are commonly connected to the support beams by arc spot (puddle) welds. These are in effect field-made plug welds. Fillet welds can also be used for the fastening of seam joints. See AWS D1.3 and other applicable AWS Standards [C.10].

2.1.4 - Mechanical fasteners. As with welded fastening, mechanical fasteners should be installed to meet construction and service load conditions.

2.1.5 - Working platform. Care must be taken to ensure that concentrated construction loads are properly distributed so as not to damage the deck. See also Section 2.2.1 of this Commentary.

2.2. Concrete

2.2.1 - Planking. The purpose of the planking is to distribute concentrated loads that exist during construction and reduce these loads to allowable levels.

2.2.2 - Concrete placement. The Standard

Practice requires that the deck be free of foreign materials prior to concrete placement because composite action depends in part on the bond between the deck and the concrete and this bond may be affected by extraneous materials. In addition, these materials may have a deleterious effect on the concrete itself. On the other hand, research has shown that tight rust aids in the surface bonding of the deck, since it increases surface roughness [C.7]. Therefore, some tight rust is considered acceptable in field applications, at least from the viewpoint of bonding. Excessive rust may, however, cause reduced deck strength and durability. Any situation contributing to continued or intermittent rusting after installation should not be allowed to exist.

During concrete placement, proper consolidation is necessary to avoid honeycombs or voids, especially in the deck corrugations and around the shear transfer devices, holes, and shear connectors. Consequently, adequate supervision and inspection are needed to ensure that the standard recommended practices are followed.

The sequence of the concrete placement must be considered during construction and location of construction joints. See Sections 2.2.5 and 2.2.6 of the Standard for the Structural Design of Composite Slabs [C.2] and the associated Commentary Sections 2.2.3, 2.2.5, and 2.2.6 [C.3]. The location of a construction joint at interior span locations (particularly at mid-span) should require that the steel deck be shored at that location. The shore supports should remain until after all concrete has been placed to prevent destroying the composite action due to deflection of the deck during the subsequent concrete placement.

The designer should specify the location of the construction joint so that it will not interfere with the desired composite support action for those cases where studs or other composite stress connectors are used. For those cases where composite shear connectors or studs are located at or near the construction joint, additional reinforcement to provide continuity may be needed. See Section 2.3.1.4 [C.3]. The removal of forms should be in compliance with the applicable sections of the ACI

standards [C.8].

2.2.3 - Shore removal. The determination of the time for removal of supporting shores may be controlled by the presence of construction loads or possible deflection limitations. If such controls exist, the removal of shores may have to occur after the concrete has reached its full f'_c compressive strength, particularly in those instances where the construction loads may be as great as the design live load. Consideration should be given to concentrated construction loads. If shoring is removed too early, more significant deflection may occur and may even result in permanent damage. The strength of the concrete during the various stages of construction should be substantiated by job-constructed and job-cured test specimens (cylinders).

2.3 - Holes and hole reinforcement

Sizable slab openings need reinforcement to distribute shears and bending moments properly around the holes. Special reinforcement may be required to prevent a possible shear-bond failure or end slip similar to that which occurs at the end of a slab. Larger openings may require reinforcement in the form of structural steel beams or supplementary reinforcing bars.

CHAPTER 3 - INSPECTION REQUIREMENTS

The strength and performance of composite steel deck slabs depends upon the fact that adequate materials be delivered to the job site. These materials must provide strengths, properties, and dimensions that are in conformance with the required capacities that are obtained in the Design Standard [C.2]; therefore, inspection requirements are needed to provide for verification of the composite slab system. The dimensions of the steel deck in conformance with the tolerances of Section C1.3, Section 1.3 of the Standard Practice [C.1], and the tested deck in Chapter 3 of the Design Standard [C.2] are very important since the design strength is based upon a certain deck shape and mechanical shear transfer device.

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- C.2. *ASCE 3-91, Standard for the Structural Design of Composite Slabs*, American Society of Civil Engineers, Steel Deck with Concrete Standards Committee, 345 East 47th Street, New York, New York 10017-2398.
- C.3. *Commentary on the Standard for the Structural Design of Composite Slabs (ASCE 3-91)*, American Society of Civil Engineers, Steel Deck with Concrete Standards Committee, 345 East 47th Street, New York, New York 10017-2398.
- C.4. *Specification for the Design of Cold-Formed Steel Structural Members*, 1986, American Iron and Steel Institute, 1101 17th Street, N.W., Washington, D.C., 20005.
Cold-Formed Steel Design Manual, American Iron and Steel Institute, Specification, March 87, Part VII - Test Procedures, Part VI Computer Aids, Commentary, Part I - Part II, Supplementary Information - Part III, Illustrative Examples - Part IV, Charts and Tables - Part V
- C.5. *Annual Book of ASTM Standards*, 1990, American Society for Testing and Materials, 1916 Race St., Philadelphia, Pa. 19103.
- (Note: Some of the ASTM Standards may have SI (metric) unit editions available. If the Standard Practice is used to build in SI units, the user shall investigate whether an SI unit edition of an ASTM standard is available.)
- ASTM A82-85 Standard Specifications for Cold-drawn Steel Wire for Concrete Reinforcement*
ASTM A370-77 Standard Method and Definitions for Mechanical Testing of Steel Products (editorial changes 1979, 1980)
ASTM A446-85 Standard Specifications for Steel Sheet, Zinc-coated (Galvanized) by the Hot-dip Process, Structural (Physical) Quality
ASTM A496-85 Standard Specifications for Deformed Steel for Concrete Reinforcement
ASTM A525-83 Standard Specifications for General Requirements for Steel Sheet, Zinc-coated (Galvanized) by the Hot-dip Process
ASTM A611-85 Standard Specifications for Steel Cold-rolled Sheet, Carbon, Structural
ASTM A185-90a Standard Specification for Steel Welded Wire Fabric, Plain, for Concrete Reinforcement
ASTM A497-90a Standard Specification for Steel Welded Wire Fabric, Deformed, for Concrete Reinforcement
ASTM A615-90 Standard Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement
ASTM A616-90 Standard Specification for Rail-Steel Deformed and Plain Bars for Concrete Reinforcement
ASTM A617-90 Standard Specification for Axle-Steel Deformed and Plain Bars for Concrete Reinforcement
ASTM A706-90 Standard Specification for Low-Alloy Steel Deformed Bars for Concrete Reinforcement
ASTM C39-86 Standard Test Methods for Compressive Strength of Cylindrical Concrete Specimens

- C.6. *Federal Specifications QQ-S775d or TT-P-641d*, Federal Supply Services, Specification Activity, General Services Administration, Washington, D.C. 20405.
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- C.8. *ACI Manual of Concrete Practice*, 1990, American Concrete Institute, Box 19150, Redford Station, Detroit, Michigan 48219.
- ACI 117-90* Standard Tolerances for Concrete Construction and Materials
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ACI 211.1-81 (Revised 1985) Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete
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- ACI 305R-77 (Revised 1982)* Hot Weather Concreting
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- C.10. *Welding Standards and Welding Handbook*. American Welding Society, 550 N.W. 42nd Ave., Miami, Florida 33126.
- AWS A5.1-81* Specifications for Carbon Steel Covered Arc Welding Electrodes
AWS A5.5-81 Specifications for Steel Low-alloy Covered Arc Welding Electrodes
AWS D1.1-90 Structural Welding Code - Steel
AWS D1.3-81 Specifications for Welding Sheet Steel and Structures
AWS D1.4-79 Structural Welding Code - Reinforcing Steel

APPENDIX A - SI UNITS

Table A.1 - SI conversion factors

	TO CHANGE	MULTIPLY BY
LENGTH	in to mm ft to mm ft to m	25.4 (exact) 304.8 (exact) 0.3048 (exact)
AREA	in ² to mm ² ft ² to m ²	645.16 (exact) 0.092903
MASS	lb to kg 2000 lb to 1000 kg lb/ft to kg/m lb/ft ³ to kg/m ³ lb/yd ³ to kg/m ³	0.453592 0.907185 1.48816 16.0185 0.593276
FORCE	lb to N kip to kN lb/in to N/m lb/ft to N/m kip/ft to kN/m psf to kN/m ²	4.44822 4.44822 175.127 14.5939 14.5939 47.880
PRESSURE	lb/in ² to kPa lb/ft ² to kPa kip/in ² to MPa	6.89476 0.04788 6.89476
SECTION MODULUS	in ³ to mm ³ in ³ /ft to mm ³ /m	16387.1 53763.5
MOMENT OF INERTIA	in ⁴ to mm ⁴ in ⁴ /ft to mm ⁴ /m	416231 1365587

APPENDIX B - DECK MEASUREMENTS

Measurements of the sheet as furnished is required rather than reference to roll tooling drawings because of variability in the manufacturing process (tooling adjustment and wear) and material variation (thickness and ductility).

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