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.

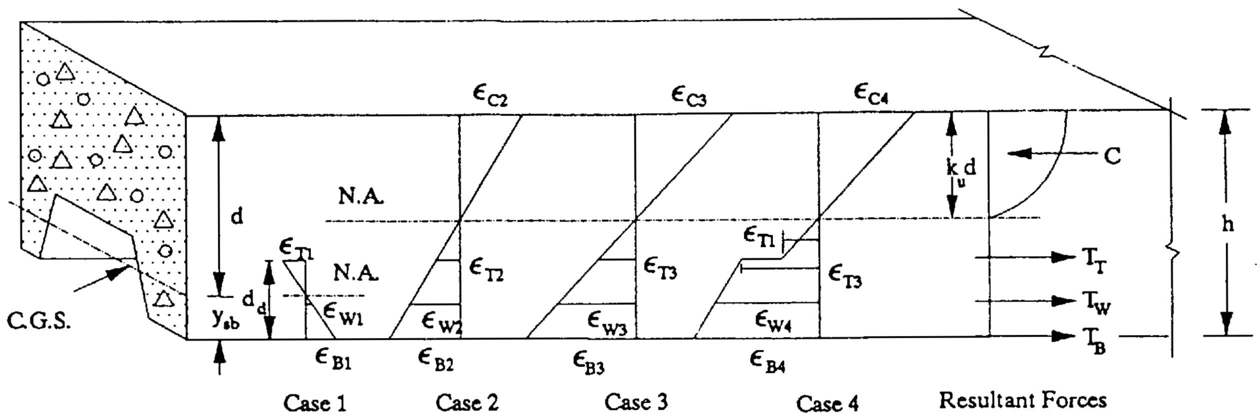
The engineer should exercise discretion when employing the general strain analysis to ensure that the proper selection of the controlling strain has been made, particularly for those instances where the deck is sufficiently deep to prevent yielding across the entire steel area. Special considerations 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

$$I_d = (I_u + I_c)/2 \qquad \text{(C.2)}$$

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 m) to 36" (900 m). 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 Lamport 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

$M_a < M_{cr}$:

$$I_e = \alpha\, I_u \tag{C.3}$$

when

$M_a \geq M_{cr}$:

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

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

for nominal d_d of 3" (76mm):

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

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_d of 1.5" (38mm):

$$\alpha = 1.0 \tag{C.6}$$

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

$$\alpha = 2.0 - 0.293 h_c \leq 1.0 \tag{C.7}$$

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

$$\alpha = 1.536 - 0.185 h_c \leq 1.0 \tag{C.8}$$

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 α = 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, $12M_u \le fS_c$ ($M \le fS_c$). 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'_f. 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.

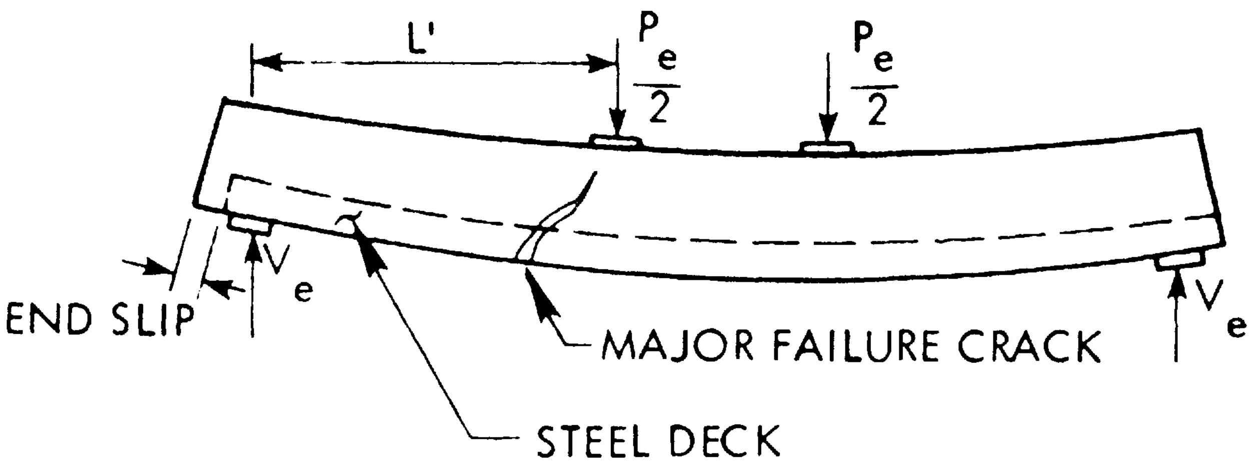


Fig. C3.1. Typical shear-bond failure

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 deliminates deck measurements. Where possible, the slab depth, h_t, 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_t should be measured at several locations in the areas where the failure is likely to occur. In this way, the h_t at the actual crack can be found easily and with sufficient accuracy. Measurement of d_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_t. 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.