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DEFLECTIONS FOR COMPOSITE STEEL DECK FLOORS

By W. B. Lamport¹ and M. L. Porter²

Keywords: Composite construction; steel-deck reinforced slabs; deflections; reinforced concrete; serviceability; one-way slabs; service design loads; concrete slabs; structural design; research; tests.

ABSTRACT: Composite steel deck floors have become a prevalent means of building floor slab construction. The increased use of higher strength concretes and shallower sections have led to more emphasis on serviceability considerations in design. A method of predicting short-term composite cold-formed steel deck deflections is proposed. Measured deflections are compared with calculated deflections using a proposed empirical procedure. The proposed method shows significant improvement over the method suggested in the Specifications for Design and Construction of Composite Slabs. Measured deflections are also compared with calculated values obtained using the method currently recommended by ACI 318-89 for normally reinforced concrete. The proposed equations are recommended to replace the procedure currently used in the Specifications for the Design and Construction of Composite Slabs.

INTRODUCTION

The instantaneous deflection behavior of noncellular steel deck panels with normal weight concrete were investigated. Examined were deflection data from 142 previously conducted shear-bond strength tests (Lamport 1984). 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 (Lamport and Porter 1990).

The data base consisted of nine span lengths ranging from six feet (1.83 m) to seventeen feet (5.18 m) and three nominal cold-formed steel-deck depths: 1-1/2" (38.1 mm), 2" (50.8 mm) and 3" (76.2 mm) from six deck manufacturers. Steel deck thicknesses ranged from 16 (1.6 mm) to 22 (0.85 mm) gage. Composite section depths were 3" (76.2 mm) to 9" (228.6 mm) with widths from 24" (0.61 m) to 36" (0.91 m). Shear transferring devices consisted of either embossments or welded wires. The concrete used for all specimens was Type 1 Portland cement with a specified

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minimum strength of 2500 (175.8 kg/cm²) to 3000 psi (210.9 kg/cm²) at 28 days. The data base distribution by deck depth, span length and gage are given in Table 1. Specimens were subjected to two-point loading with varying shear spans, (L') as shown in Fig. 1.

CURRENT CODE RECOMMENDATIONS

Specifications for the Design and Construction of Composite Slabs (ASCE Standard 1984) recommends a simple average for the composite effective moment of inertia, I_d for deflection calculations at service design loads.

$$I_d = 1/2(I_u + I_{cr}) \quad (1)$$

I_d was based on deflection data from specimens used to determine the shear-bond characteristics of composite steel deck sections (Porter and Ekberg 1976).

Figure 2 shows a typical load-deflection curve of a long 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. 1 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. 66" (1676.4 mm) span group deflections were under estimated by approximately 13% whereas for the 198" (5029.2 mm) span specimens the deflections were underestimated by an average of 46%.

ACI RECOMMENDED I_e FOR NORMALLY REINFORCED CONCRETE SLABS

The deflection procedure recommended by the American Concrete Institute (ACI) in Building Code Requirements for Reinforced Concrete (ACI 318-89, 1989) was developed by Branson (Branson 1965). The method is based on an elastic analysis of stresses and strains on a cracked reinforced concrete section using modular ratio theory. To account for the tension stiffening effect of the uncracked concrete in the tension zone, and variation in I along the beam span, Branson suggested an empirical equation for I_e as

$$I_e = (M_{cr}/M_a)^3 I_g + \{1 - (M_{cr}/M_a)^3\} I_{cr} \leq I_g \quad (2)$$

The range of I_e is from I_g when M_a is below M_{cr} to I_{cr} as M_a approaches overload. Equation 2 was developed from a statistical study of test specimens which had M_{max}/M_{cr} values from 2.2 to 4.0 and I_g/I_{cr} ratios from 1.35 to 3.5. For these limits, Eq. 1 provides a reasonable estimate of the average stiffness of a beam. The accuracy of Eq.1 is approximately $\pm 25\%$ (ACI 435 1972) for service loads.

RECOMMENDED I_e FOR COMPOSITE STEEL-DECK SLABS

The proposed criteria is based on work by Lamport and Porter (Lamport 1984). The method is similar in format with the method currently recommended by ACI 318-89 for normally reinforced concrete (ACI 318-89 1989). The method recommends an effective moment of inertia approach in which the moment of inertia varies between kI_u when the applied moment is below the cracking moment, M_{cr} , and approaches the moment of inertia of the steel deck taken about the composite neutral axis, I_D , as the load approaches ultimate. The final recommended form of I_e is as follows.

$$M_a < M_{cr}: \quad I_e = kI_u \quad (3)$$

$$M_a \geq M_{cr}: \quad I_e = kI_u(M_{cr}/M_a)^m + \{1 - (M_{cr}/M_a)^m\}I_D \leq kI_u \quad (4)$$

where the power function m is ;

$$d_d \leq 2"; \quad m = 0.55 \quad (5)$$

$$d_d = 3"; \quad m = 1.3 \quad (6)$$

The stiffness reduction coefficient, k was included to reduce the initial stiffness below I_u . Of the specimens which had load levels (added dead load due to shore removal, superimposed dead load of loading apparatus, and applied live load) below the cracking moment, M_{cr} , over 90% had initial stiffnesses below I_u . Though the loads were not theoretically sufficient to cause the moment in the specimen to reach M_{cr} . An analysis of the data indicated that the stiffness reduction was related to the thickness of the concrete above the steel deck, t_c . The ratio of I_{exp}/I_u decreased from an approximate mean of 0.90 to 0.65 as t_c increased from 3.4" (86.4 mm) to 5.1" (129.5 mm).

Results of a linear regression analysis, indicated the stiffness reduction coefficient, k should be obtained using Eq. 7, 8 and 9 for 1-1/2" (38.4 mm), 2" (50.8 mm) and 3" (76.2 mm) nominal steel deck depths, respectively.

Values of k for $3.4" \leq t_c \leq 5.1"$:

$$d_d = 1-1/2"; \quad k = 1.0 \quad (7)$$

$$d_d = 2"; \quad k = 2.0 - 0.293t_c \leq 1.0 \quad (8)$$

$$d_g = 3";$$

$$k = 1.536 - 0.185t_c \leq 1.0 \quad (9)$$

Equations 7 - 9 were determined for the ranges of t_c given. The upper limit of $t_c = 5.1"$ (129.5 mm) should include most slab depths used in normal construction. For values of $t_c > 5.1"$ (129.5 mm) the authors recommend the value of k be determined using $t_c = 5.1"$. For slab depths with $t_c < 3.4"$ (86.4 mm), the authors suggest that a value of $k = 1.0$ be used.

Figure 3 compares the current method, Eq. 1, the ACI 318-89 approach, for normally reinforced concrete, Eq. 2 and the proposed method Eqs. 3 - 9. The figure shows the cumulative percentage of calculated deflections which are within a given error range of the measured deflections for a typical midspan specimen. The figure indicates that proposed Eqs. 3 - 9, yield substantial improvement over Eq. 1. This improvement is even more pronounced in the longer spans where Eq. 1 is not capable of accounting for the extensive cracking that occurs. Equations 3 - 9 are also more conservative in the prediction of the experimental deflections for loads at and below service design load when compared to Eq. 1. Table 2 compares calculated deflections using Eq.1 and Eqs. 3 - 9 with measured values for selected tests values. The deflection values given correlate with the experimental deflections closest to service design load, where service design live load for a given specimen was determined by Eq. 10, (ASCE Standard 1984).

$$LL = \sqrt{[2V_u/L - 1.4(\gamma W_1 + W_3)]/1.7} \quad (10)$$

The factor γ accounts for the amount of the self weight of the slab in composite action, i.e. the amount of the load applied to the composite system after shore removal. For the shoring at ends only $\gamma = 0.0$, shored at ends and center $\gamma = 0.625$ and for the entire deck length shored $\gamma = 1.0$.

CONCLUSIONS

- The proposed effective moment of inertia approach for calculating instantaneous deflections of composite floor slabs given by Eqs. 3 - 9 were found to give reliable predictions of deflections for the spans, steel-deck depths and steel-deck types used in the investigation.
- The deflection predictions of the proposed method were usually more conservative than those given by Eq. 1, for loads at or below service design load.
- The proposed method Eqs. 3 - 9, unlike the current method Eq. 1, is more able to account for the extensive cracking that occurs in the longer spans, by the inclusion of the (M_{cr}/M_a) term.

- The format of the proposed method is familiar to the designer in that it closely resembles the approach used by ACI 318-89 for normally reinforced concrete.
- The authors recommend that Eqs. 3 - 9 be used for deflection design computations involving slabs reinforced with cold-formed steel decking.

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APPENDIX I.-REFERENCES

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APPENDIX II.-NOTATION

The following symbols are used in this paper:

d_d	Overall depth of steel deck profile, in.
I_{cr}	Moment of inertia of composite section based on cracked section, in ⁴ /ft of width
I_d	Moment of inertia of composite section considered effective for deflection computations, in ⁴ /ft of width
I_D	Moment of inertia of steel deck about its centroidal axis, in ⁴ /ft width
I_e	Effective moment of inertia considering the effect of cracking, in ⁴ /ft width
I_{exp}	Experimental moment of inertia of section, in ⁴ /ft of width
I_g	Moment of inertia of gross section about centroidal axis, neglecting reinforcement, in ⁴ /ft of width
I_u	Moment of inertia of composite section based on cracked section, in ⁴ /ft of width
k	Stiffness correction factor
L	Length of span, in
LL	Allowable superimposed live load for service conditions, psf
M_a	Maximum moment in member at stage deflection is computed, in lbs/ft of width
M_{cr}	Cracking moment, in lbs/ft of width
t_c	Depth of concrete above top corrugation of steel deck, in
V_e	Maximum experimental shear at failure obtained from laboratory tests (not including weight of slab), lbs/ft of width
W_1	Weight of slab ($DL_d + DL_c$), psf
W_3	Dead load applied to slab, exclusive of W_1 , psf
γ	Coefficient for proportion of dead load added upon removal of shore

TABLE 1 Depth, Span and Gage of Cold-Formed Steel Decks

Quantity	Depth	Span	Gage	Quantity	Depth	Span	Gage
	(in)	(in)			(in)	(in)	
4	1-1/2	92	20	10	3	90	16
4	1-1/2	140	16	6	3	90	18
2	1-1/2	140	18	5	3	90	20
36	1-1/2	140	20	2	3	90	22
4	1-1/2	140	22	6	3	138	16
7	2	66	18	7	3	138	18
6	2	66	20	3	3	138	20
5	2	114	18	2	3	138	22
3	2	114	20	3	3	186	18
6	2	162	18	5	3	186	20
2	2	162	20	8	3	198	16
				3	3	198	18
				3	3	198	22

(25.4 mm = 1 in)

TABLE 2 Comparison of Calculated Deflections with Measured Values for Deflections Closest to Service Design Load

Deck Depth	Span	Test No.	Measured	Calculated		Meas./Calc.	
				Eq. 1	Eqs. 3-9	Eqs. 1	Eqs. 3-9
(in)	(in)		(in)	(in)	(in)		
1-1/2	140	055	0.21	0.22	0.22	0.95	0.97
2	114	422	0.07	0.08	0.08	0.83	0.88
2	114	445	0.17	0.16	0.18	1.01	1.12
2	162	449	0.84	0.62	0.74	1.35	1.13
2	162	462	0.55	0.45	0.50	1.21	1.10
3	138	364	0.12	0.11	0.11	0.94	0.97
3	138	432	0.08	0.09	0.09	1.10	1.21
3	198	376	0.67	0.41	0.58	1.64	1.16
3	198	385	0.68	0.41	0.60	1.65	1.13
3	198	411	0.58	0.30	0.42	1.97	1.40

(25.4 mm = 1 in)

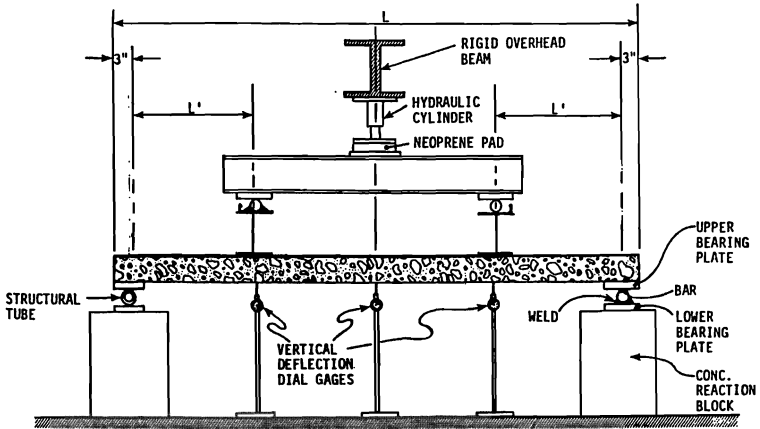


FIGURE 1. Typical Specimen Subjected to Two-point Loading

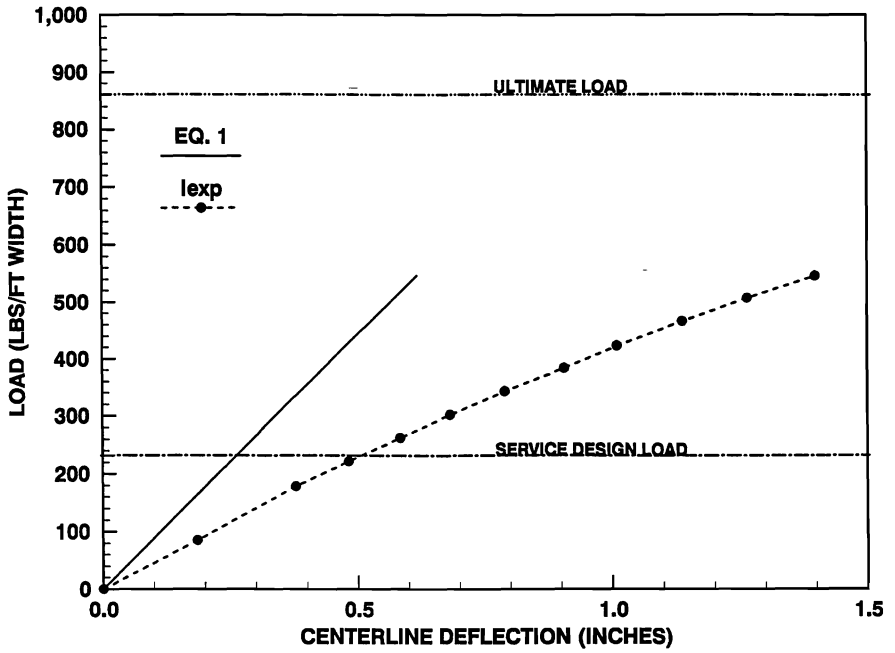


FIGURE 2. Comparison of Measured and Calculated Deflections Using Eq. 1 for 198" Span Specimen No. 411

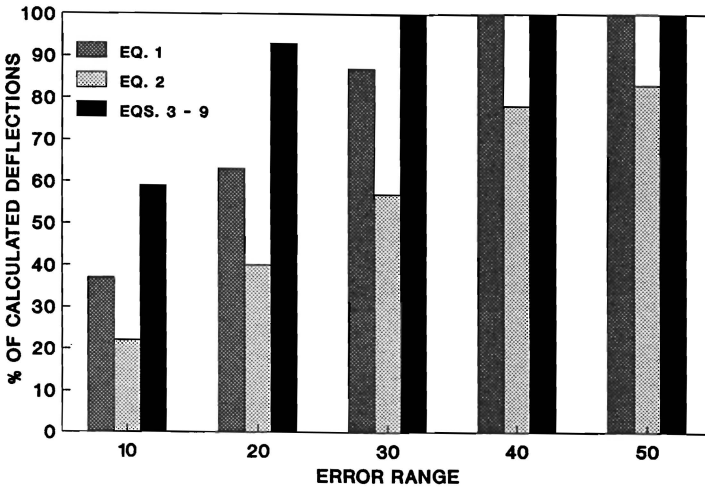


FIGURE 3. Comparison of Calculated and Measured Deflections for 114" Span Specimen with 2" Nominal Steel Deck