CHAPTER I

Reliability Assessment of Composite Beams

J.R. Ubejd Mujagić[1] and W.S. Easterling[2]

to be submitted to the ASCE Journal of Structural Engineering

ABSTRACT: Composite steel-concrete flexural members have become increasingly popular in design and construction of floor systems, structural frames, and bridges. Constant research advances have resulted in numerous enhancements and changes to the American design practice, as embodied in the composite construction provisions of AISC Specification. This paper presents results of a comprehensive reliability study of composite beams. The study considers Specification changes since the 1976 reliability study by Galambos and Ravindra, considers a larger database of experimental data, and evaluates recent proposals for changes in the design of shear connectors. Comparison of three different design methods is presented based on 15,064 composite beam cases. A method to consider the effect of the degree of shear connection on the strength reduction factor is proposed.

CE Database keywords: composite beams, steel-concrete composites, shear connection, studs, reliability, AISC Specification.

I.1 INTRODUCTION

Composite steel-concrete flexural members have become increasingly popular in design and construction of floor systems, structural frames, and bridges. While in some form, composite construction has been present throughout most of the past century, especially in bridge construction, the usage of composite members has dramatically increased since the 1970s due to introduction of formed steel deck and concurrent advances in the field gained from experimental and analytical research. As a consequence of ongoing research, American design practice, as embodied in the AISC design specifications, has undergone constant revision and enhancement. The composite design provisions of the current LRFD AISC Specification (AISC 1999) incorporate several new changes, especially as they relate

[1] Structural Engineer, Pinnacle Structures, Inc., Cabot, AR, [2]Professor, Dept. of Civil & Environmental Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA

to the design of shear connection. Based on new findings (Yuan 1996, Roddenberry et al. 2002a,b) further revisions are possible. This paper presents results of a comprehensive reliability study which focuses on the effect of shear connection on the flexural strength reduction factor. Three strength calculation models are considered. The first is that by Grant et al. (1977), which is the basis of the AISC Specification provisions, and is in this study referred to as 1999 AISCS. The second model considered is that proposed by Roddenberry et al. (2002a,b), (R), and the third one is a simplified version of Roddenberry's model, referred to herein as Roddenberry Simplified (RS). It should be noted that the main focus of the paper is flexural members with either solid slabs, or ribbed slabs with the ribs oriented perpendicular to the member. While not specifically addressed herein, design procedures for composite beams with ribs oriented parallel to member could be reviewed in light of this work given the inherent behavioral similarities. Focus is also made on positive bending members utilizing headed studs as a mean of shear connection.

The strength reduction factor, ϕ, for design moment strength of composite beams was first computed in the reliability study by Galambos and Ravindra (1976) based on the design procedure used at that time. The procedure has since undergone significant revisions, especially with respect to design of shear connection in slabs with formed deck. The currently used model for beams with formed deck was derived by Grant et al. (1977), and was found in many cases to be inaccurate and unconservative (Johnson 1994, Easterling et al. 1993, Yuan 1996, Roddenberry 2002a,b). Several modifications to this model were made in the current draft of Eurocode 4 (CEN 1992) to address these deficiencies. However, the issues of inaccuracy and unconservative strength prediction largely remain unaddressed in 1999 AISCS. Arguments were also made that the strength reduction factor could be a function of the degree of shear connection in a composite beam (Mujagic et al. 2001). This concept is illustrated using Fig. I.1. Depicted in that figure is a generalized plot of the relationship between the composite beam moment strength and the degree of shear connection in the beam. One can consider a generalized distribution of shear connection strength (R_S), as illustrated in the figure. It is apparent that the same range of R_S will result in a higher range of flexural strengths $\left(R_{M_{2-3}}\right)$ for beams designed with a low degree of shear connection $(M_{25\%})$, then will be the case for the range $(R_{M_{100\%-1}})$ that corresponds to beams designed with a higher degree of shear connection $(M_{100\%})$.

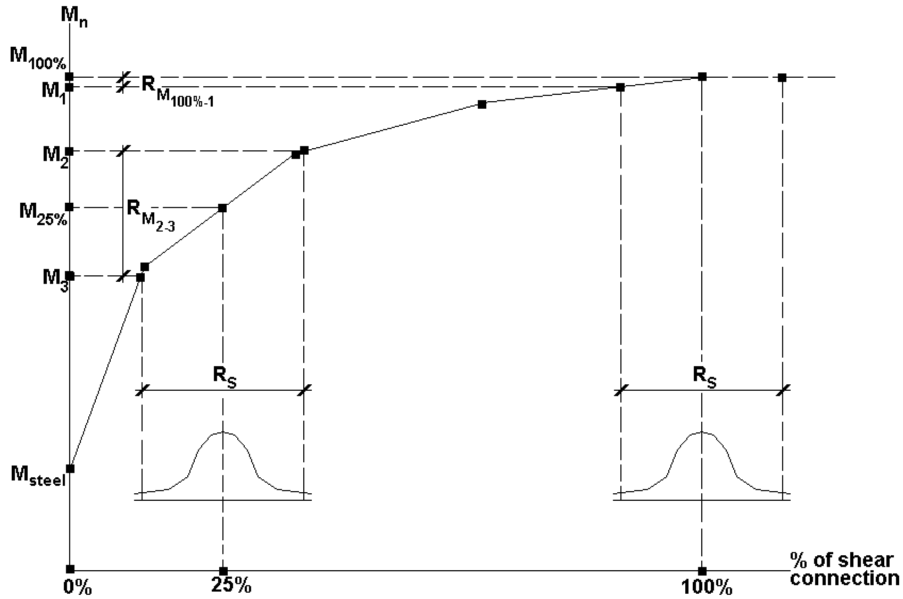

Fig. I.1 Effect of Degree of Shear Connection on Moment Strength

Following the review of current and proposed design practice, this paper presents results of an incremental reliability analysis, by which the strength reduction factor is calculated for various ranges of partial shear connection. The analyses were performed taking advantage of the fact that the location of the plastic neutral axis (P.N.A.) in composite beams can be related to the degree of shear connection for most practical configurations. Typically, a P.N.A. located in the beam web corresponds to a relatively low degree of shear connection, generally below 40%. A P.N.A. location in the flange corresponds to higher degree of shear connection, and a P.N.A. at or above the top of the steel section corresponds to a fully composite beam. The analyses were performed for all three models of shear connection strength. Analyses were performed separately for beams with ribbed slabs and solid slabs, and results of analyses that combine both types of tests are also presented. A discussion on the effects of various parameters is also included.

Finally, design specifications generally stipulate a uniform ϕ when determining the design strength for a particular limit state. Recognizing the complexity of determining a uniform ϕ for composite beams and all the variations and possibilities involved, this paper provides an alternative method for determining ϕ, based on the degree of shear connection in the member. The approach is analogous to that stipulated by ACI-318 for design of beam-columns, where the resistance factor is a function of the presence of compression and bending in the member and their respective proportions of the member capacity (ACI 2002).

I.2 REVIEW OF CURRENT DESIGN PRACTICE AND PROPOSED MODIFICATIONS

The 1999 AISC Specification stipulates that one of the ways that the composite beam flexural strength is computed is from a plastic stress distribution on the composite cross-section. By this approach, the steel section is assumed to be fully yielded and concrete has reached the stress of $0.85f'_c$ up to certain depth of the slab. The stress diagram components, as illustrated in Fig. I.2, consist of a steel beam and the concrete slab. Effective width of the slab on each side of the beam center-line is determined as the smallest of: 1/8th of beam span (center-to-center of supports), one-half the distance to the center-line of the adjacent beam, the distance to the edge of the slab (AISC 1999). The flexural strength, as given in the AISC Commentary to the Specification, is determined from Eq. I.1.

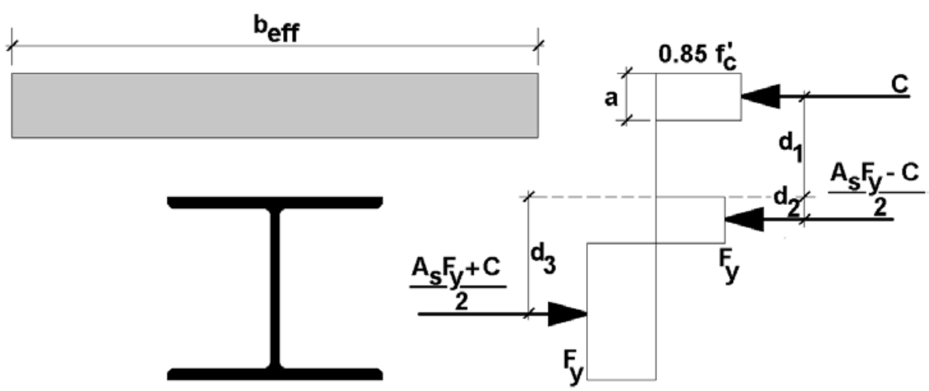

Fig. I.2 AISC Flexural Model

$$M_n = \frac{A_sF_y + C}{2}(d_3 - d_2) + C(d_1 + d_2) \tag{I.1}$$

where:

A_s = area of beam cross-section, mm^2

F_y = yield stress strength, N/mm^2 = MPa

C = $\min\{A_s F_y, 0.85 f'_c b_{eff} A_c, \Sigma Q_n\}$, N

f'_c = concrete compressive strength, N/mm^2

b_{eff} = effective slab width, mm

A_c = $(b_{eff})(t_s)$, mm^2

ΣQ_n = design strength of shear connection, N

The plastic stress distribution must be adjusted to account for incomplete interaction across the steel-concrete interface, as exhibited in partially composite beams. The stipulated resistance factor is 0.85.

The calculation of shear connection strength is based on the model proposed by Grant et al. (1977) and has been a part of the specification for several editions. The nominal strength of a shear stud is determined using Eq. I.2.

$$Q_n = 0.5 A_{sc} \sqrt{f'_c E_c} \leq A_{sc} F_u \qquad \text{(Eq. I.2)}$$

where:

A_{sc} = area of stud cross-section, mm^2

E_c = concrete modulus of elasticity, N/mm^2

= $w_c^{1.5}(0.043)\sqrt{f'_c}$

w_c = concrete unit weight, kg/m^3

f'_c = concrete compressive strength , N/mm^2

F_u = ultimate tensile strength stress for studs, N/mm^2

If the slab has ribs perpendicular to the beam section, the strength of a stud is computed by multiplying the value obtained using Eq. I.2 by the following strength reduction factor:

$$SRF = \frac{0.85}{\sqrt{N_r}}(w_r / h_r)[(H_s / h_r) - 1.0] \leq 1.0 \qquad \text{(Eq. I.3)}$$

Parameters given in Eq. I.3 are depicted in Fig. I.3. In Eq. I.3, N_r may not exceed 3 in computations, and H_s may not exceed the value of h_r + 76 mm. Also, the upper limit on SRF is reduced from 1.0 to 0.75 where there is only a single stud placed in a rib. This limit was a new addition to the 1999 AISC Specification and was intended to be a temporary provision in response to numerous laboratory findings suggesting that the strength of a single stud in a rib found by Eqs. I.2 and I.3 may be unconservative for many deck profiles (Easterling et al. 1993).

The strength of shear studs where slab ribs are oriented parallel to the member length is computed by using Eq. I.2. The strength obtained using Eq. I.2 must be multiplied by the SRF shown in Eq. I.4 if $w_r/h_r<1.5$.

$$SRF = 0.6(w_r / h_r)[(H_s / h_r) - 1.0] \leq 1.0 \qquad \text{(Eq. I.3)}$$

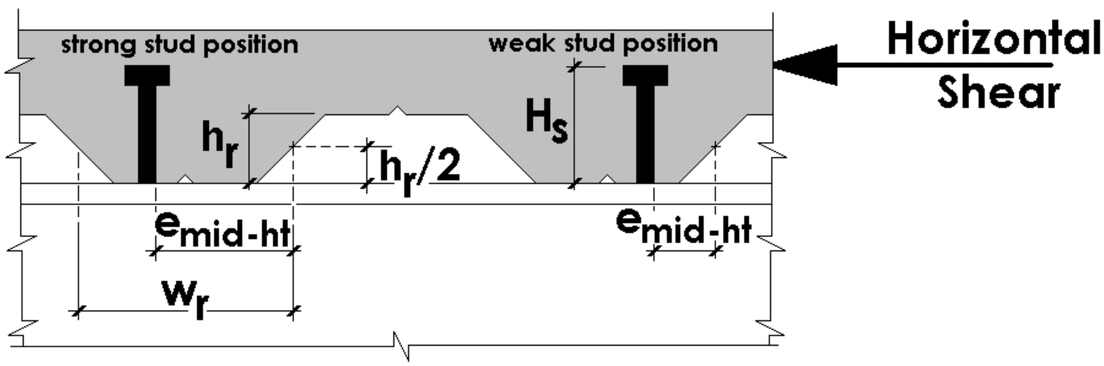

Fig. I.3 Slab and Stud Geometric Parameters

While there was no evidence that Eq. I.4 leads to either inaccurate or unconservative results, a number of authors have determined that revisions to or replacements of Eqs. I.2 and I.3 are warranted (Johnson 1994, Easterling et al. 1993, Yuan 1996, Roddenberry 2002a,b). The motivation for this is in several reasons. As noted earlier, the model by Grant et al. (1977) was found to be unconservative when predicting single-stud rib connections. Further, it does not account for the loss of strength caused by placing studs on the weak side of the rib, which is illustrated in Fig I.3. The model was also found to be unconservative when studs are welded to the beam directly, rather than through the decking. It overestimates the strength where strong studs are placed in relatively weak ribs and is found to be generally inconsistent and inaccurate in number of other applications (Johnson and Anderson 1993). Finally, it is thought that the methodology used to derive the model may not have been appropriate. Namely, the model by Grant et al. (1977) was developed by back-calculating shear connection strengths from full-scale composite beam tests. As shown in Fig. I.1, if the degree of shear connection is relatively high, significant error in predicting its strength may not be obvious. This leads to difficulty in assessing the strength and behavior of shear connection solely based on full-scale test, and such evaluation should be based on push-out tests.

The model by Roddenberry et al. (2002a,b) would lead to elimination of Eqs. I.1 and I.2. The proposed model is shown as Eq. I.5. It considers new parameters found to have effect on strength such as, location of the studs within the rib, decking thickness, and ratio of stud diameter to top flange thickness.

For <u>*h_r* = 51mm or 76 mm</u>, with $D/t_f \leq 2.7$:

$$Q_n = R_p R_n R_d A_{sc} F_u \qquad \text{(I.5a)}$$

R_p = 0.68 for $e_{mid\text{-}ht.} \geq 56$ mm (strong position studs)

= 0.48 for $e_{mid\text{-}ht.} < 56$ mm (weak position studs)

= 0.52 for staggered position studs

R_n = 1.0 for one stud per rib or staggered position studs

= 0.85 for two studs per rib

R_d = 1.0 for all strong position studs

= 0.88 for 22 gauge deck (weak studs)

= 1.0 for 20 gauge deck (weak studs)

= 1.05 for 18 gauge deck (weak studs)

= 1.11 for 16 gauge deck (weak studs)

For <u>*h_r* = 25 mm or 38 mm</u>, with $D/t_f \leq 2.7$:

$$Q_n = R_n 13700 e^{(A_{sc} F_u / 92675)} \qquad \text{(I.5b)}$$

R_n = 1.0 for one stud per rib

= 0.85 for two studs per rib

For <u>*h_r* = 51 mm or 76 mm</u>, with $D/t_f > 2.7$:

$$Q_n = R_p R_n R_d A_{sc} F_u - 6700\left(\frac{D}{t_f} - 2.7\right) \qquad \text{(I.5c)}$$

For *h_r = 25 mm or 38 mm*, with D/t_f > 2.7:

$$Q_{sc} = R_n 13700 e^{(A_{sc}F_u / 92675)} - 6700\left(\frac{D}{t_f} - 2.7\right) \quad \text{(I.5d)}$$

where:

D = stud diameter, mm

t_f = flange thickness, mm

The above given method can be simplified by retaining the left side of Eq. I.2, and applying several modifications to its right side. The Simplified Roddenberry model is shown as Eq. I.6. The motivation to simplify the original model is to retain the previous recognizable form and make it more acceptable for codification.

$$Q_n = 0.5A_{sc}\sqrt{f_c^{'}E_c} \leq R_g R_p A_{sc} F_u \quad \text{(Eq. I.6)}$$

where:

R_g = 1.0 for one stud welded in a steel deck rib with the deck oriented perpendicular to the steel shape; for any number of studs welded in a row directly to the steel shape; for any number of studs welded in a row through steel deck with the deck oriented parallel to the steel shape and the ratio of the average rib width to rib depth ≥ 1.5

= 0.85 for two studs welded in a steel deck rib with the deck oriented perpendicular to the steel shape; for one stud welded through steel deck with the deck oriented parallel to the steel shape and the ratio of the average rib width to rib depth < 1.5

= 0.7 for three or more studs welded in a steel deck rib with the deck oriented perpendicular to the steel shape

R_p = 1.0 for studs welded directly to the steel shape (i.e. not through steel deck or sheet) and having a haunch detail with not more than 50% of the top flange covered by deck or sheet steel such as girder fillers.

0.75 for studs welded in a composite slab with the deck oriented perpendicular to the beam and $e_{mid\text{-}ht}$ ≥ 51 mm; for studs welded through steel deck, or steel sheet used as girder filler material, and embedded in a composite slab with the deck oriented parallel to the beam

0.6 for studs welded in a composite slab with deck oriented perpendicular to the beam and $e_{mid\text{-}ht} < 51$ mm

I.3 EVALUATION OF STRENGTH REDUCTION FACTOR ϕ FOR COMPOSITE BEAMS WITH RIBBED AND SOLID SLABS

The resistance factors were determined using a first-order, second-moment, probabilistic method. This simplified method uses two statistical pieces of information, means and coefficients of variation (C.O.V.). Reliability (β) is the relationship between these two measures (Galambos and Ravindra 1973). This method was first developed by Cornell (1969) and Ravindra et al. (1969). The LRFD criteria and ϕ for composite beams were developed by Galambos and Ravindra (1976), and Hansell et al. (1978). The criteria used to calculate resistance factors for steel structures and composite beams are defined in detail by Galambos and Ravindra (1973, 1976), and concepts used in this study are outlined below. The resistance factor was computed using Eq. I.7.

$$\phi = \frac{R_m}{R_n} e^{(-\alpha \beta V_R)} \qquad \text{(Eq. I.7)}$$

R_m/R_n is the average ratio of tested to predicted strength and is derived directly from comparing the tested to predicted strengths as noted below. Consistent with previous work of Galambos and Ravindra (1973), values of $\alpha = 0.55$ and $\beta = 3.0$ are used in this study. The former was found to be the proper separation coefficient between load and resistance effects, while the latter was derived based on calibration with simple compact beams to achieve a similar level of safety (Galambos and Ravindra 1976). The coefficient of variation (C.O.V.) for the resistance, V_R, is shown as Eq. I.8.

$$V_R = \sqrt{V_M^2 + V_F^2 + V_P^2 + V_M^2 V_F^2 + V_M^2 V_P^2 + V_F^2 V_P^2 + V_M^2 V_F^2 V_P^2} \qquad \text{(Eq. I.8)}$$

Eq. I.8 is generally simplified to exclude the product terms. Those terms, however, are retained in this study, as statistical variation of shear connection is shown to be considerable. V_F represents the C.O.V. for fabrication and was taken as 0.05. V_P represents the C.O.V. in design theory and is derived directly from comparison of theoretical calculations and tested data. Statistical characteristics of beam flexural strengths were calculated using 65 solid-slab tests and 87 ribbed-slab tests compiled from various resources. The summary of test parameters and calculated values are shown in Tables I.1 through I.3. Statistical characteristics of shear connectors were computed using push-out test database gathered by Rambo-Roddenberry et al. (2002a). The tests considered include 254 push-out tests performed by Rambo-Roddenberry et al. (2002a), Lyons et al. (1994), Sublett et al. (1992), and Diaz et al. (1998). Various values of V_P derived in this study for beams and shear connectors are shown in Tables I.4 and I.5. V_M represents the C.O.V. for material properties. Its value was determined using Eq. I.8 and known statistical properties of the materials as noted below.

$$V_M = \frac{\sigma_M}{R_m} \qquad \text{(Eq. I.9)}$$

In the Eq. I.9, the parameter σ_M, represents the standard derivation in the material strength. Its value is easily extracted from the test data when a single material is involved. Its determination becomes more complex when multiple materials are a part of the resistance equation, as is the case in this study and given by Eq. I.10. In the Eq. I.0, the parameter K_n represents a generalized value of a particular strength parameter under consideration. Consistent with earlier studies (Galambos and Ravindra 1973, 1976), it was considered that the geometric properties, such as A_s, t_s, d, and b_{eff} are deterministic, and that any actual variation therein is accounted for in the value of V_F. Further, all variables involved in determining the strength of shear connectors and the moment strength of beams were considered statistically independent.

$$\sigma_M^2 = \left(\frac{\partial R}{\partial K_1}\right)_m^2 \left(\sigma_{K_1}\right)^2 + \left(\frac{\partial R}{\partial K_2}\right)_m^2 \left(\sigma_{K_2}\right)^2 + \ldots\ldots + \left(\frac{\partial R}{\partial K_n}\right)_m^2 \left(\sigma_{K_n}\right)^2 \qquad \text{(Eq. I.9)}$$

Statistical characteristics of the material properties for steel beams and concrete were taken from Galambos et al. (1978). The values shown represent means of the properties of flanges and webs of hot-rolled shapes. Statistical characteristics of concrete properties were taken from Galambos and Ravindra (1976), and statistical characteristics of headed stud properties were calculated in this study using test data from Nelson Stud Welding Company (1971a,b, 1972). Statistical properties of steel are largely based on evaluation of A36M, and A572M material databases. However, the recent study by Bartlett et al. (2003) shows that consideration of A992M steel does not require significant revisions to the statistical characteristics of the original database. The material properties are as follows:

Steel Beams: $(F_y)_m = 1.08F_y$, $V_{F_y} = 0.11$, $\sigma_{F_y} = (F_y)_m V_{F_y} = 0.12F_y$

Concrete: $(f'_c)_m = 1.17f'_c$, $V_{f'_c} = 0.22$, $\sigma_{f'_c} = 0.26f'_c$

Headed Shear Studs: $(F_u)_m = 1.14F_u$, $V_{F_u} = 0.09$, $\sigma_{F_u} = 0.10F_u$

To simplify the statistical analysis, a built-up I-beam was considered. Further, the flexural strength expression was written considering the presence of a solid slab. These simplifications have no significant impact on the final outcome of the analysis, as long as proper statistical parameters for shear connection and ribbed slab beam tests are substituted where appropriate. The total of 15,064 computations of resistance factor were performed, both with and without consideration of influence of shear connector strength. In 2,002 of those cases, the P.N.A. was in the concrete slab, in 6,006 cases it was in the steel flange, while in 7,056 cases P.N.A. occurred in the web. Consideration was given both to cases in which the strength is governed by the shear connection, and those in which it is governed by the weaker of concrete slab and the steel beam. All computations are given by Mujagic and Easterling (2003). The analysis shown below illustrates how the required components of Eq. I.7 are computed. The approach used is similar to that employed by Galambos and Ravindra (1976).

Table I.1 Composite Beams with Formed Deck – Test Parameters

Authors	Beam Test	Section	L (m)	b_{eff} (mm)	t_s (mm)	h_r (mm)	w_r (mm)	D (mm)	H_s (mm)	N[a, b]	e_{mid-ht} (mm)[k]	Deck Ga.	w_c (kg/m^3)	f'_c (MPa)	F_{yf} (MPa)[c]	F_{yw} (MPa)[c]	F_u (MPa)[j]
Grant et al. (1977)	1A1R	W16x40	7.32	1829	102	38	57	19	76	10(2), 4(1)	19, 19	20	1836	24	474	516	448
	1A2	W16x40	7.32	1829	114	51	76	19	89	10(2), 4(1)	29, 29	20	1889	26	396	425	448
	1A3R	W16x40	9.75	2438	140	76	114	19	114	13(2)	48	20	1842	22	452	470	448
	1A5R	W16x45	6.10	1524	102	38	76	19	76	8(2), 2(1)	29, 29	20	1889	26	456	471	448
	1A6R	W16x45	7.32	1829	114	51	102	19	89	10(2)	41	20	2019	34	456	465	448
	1A7	W16x40	9.75	2438	140	76	152	19	114	7(2), 4(1)	67, 67	20	1913	29	436	454	448
	1B1	W16x58	9.75	2438	140	76	114	19	114	12(2), 1(1)	48, 48	20	1967	26	222	241	448
	1B2	W16x58	7.32	1829	102	38	76	19	76	9(2), 1(1)	29, 29	20	1844	33	256	270	448
	1C1	W16x40	7.32	1829	102	38	57	19	76	11(1)	19	20	1868	30	398	430	448
	1C2A	W16x40	9.75	2438	140	76	114	19	114	2(2), 5(1)	48, 48	20	1815	28	455	463	448
	1C2B	W16x40	9.75	2438	140	76	114	19	114	5(2), 2(1)	48, 48	20	1815	28	456	476	448
	1C3	W16x40	7.32	1829	114	51	102	19	89	8(1)	41	20	1887	33	477	515	448
	1C4	W16x45	9.75	2438	140	76	152	19	114	7(2)	67	20	1902	22	447	470	448
	1D1	W16x40	7.32	1829	102	38	57	19	76	10(2), 4(1)	19, 19	20	1839	24	476	509	448
	1D2	W16x40	7.32	1829	102	38	57	19	76	10(2), 4(1)	19, 19	20	1844	32	382	418	448
	1D3	W16x45	9.75	2438	140	76	152	19	114	7(2), 4(1)	67, 67	20	1961	27	447	468	448
	1D4	W16x40	9.75	2438	140	76	152	19	114	7(2), 4(1)	67, 67	20	1999	33	443	474	448
Robinson et al. (1971)	71-17(A1)	W12x19	6.40	1600	102	38	57	19	76	6(1)	19	18	2323[d]	30	287	322	448
	71-17(A2)	W12x19	6.40	1600	102	38	57	19	76	6(2)	19	18	2323[d]	39	287	322	448
	71-17(A3)	W12x19	6.40	1600	102	38	57	19	76	9(1)	19	18	2323[d]	39	281	319	448
	71-17(A4)	W12x19	6.40	1600	102	38	57	19	76	6(2)	19	18	2323[d]	27	281	319	448
	71-17(A5)	W12x19	6.40	1600	127	38	57	19	102	6(2)	19	18	2323[d]	27	281	319	448
	71-17(B1)	W12x19	6.40	1600	102	38	57	19	76	21(1)	19	18	2323[d]	32	284	320	448
	71-17(B2)	W12x19	6.40	1600	102	38	57	19	76	11(1)	19	18	2323[d]	24	284	320	448
	71-17(B3)	W12x19	6.40	1600	102	38	57	19	76	16(1)	19	18	2323[d]	24	284	320	448
	71-17(B4)	W12x19	6.40	1600	102	38	57	19	76	21(1)	19	18	2323[d]	24	284	320	448
Michalski et al. (1970)	70-4	W16x26	9.14	1778	102	38	54	19	76	14(1)[d]	18[d]	20[d]	1815[e]	22	272	292	448
Poletto et al. (1970)	70-3(A)	W12x27	4.57	1143	140	38	49	19	114	(9)1, 3(2)	15, 15[d]	20[d]	1890[e]	33	243	272	448
	70-3(B)	W12x27	4.57	1143	140	38	49	19	114	(9)1, 3(2)	15, 15[d]	20[d]	1890[e]	33	243	272	448
Jones et al. (1966)	66-11(42)	W8x15	6.10	1118	102	38	57	19	76	4(1)	19[d]	20[d]	2323[d]	23	342[f, g]		448
	66-11(56)	W8x15	6.10	1118	102	38	57	19	76	5(1)	19[d]	20[d]	2323	23	319[f, g]		448
	66-11(W)	W14x30	6.10	1118	102	38	57	19	76	13(1)	19[d]	20[d]	2326	28	271[f, g]		448
Slutter et al. (1964)	64-15(H1)	W12x27	4.57	1143	102	38	114	19	76	12(1)	48[d]	20[d]	1842[d]	27	258	314	448
	64-15(E1)	W12x27	4.57	1143	102	38	114	19	76	5(1), 4(2)	48, 48[d]	20[d]	1842[d]	28	237	271	448
	64-15(E2)	W12x27	4.57	1143	102	38	114	19	76	8(2)	48[d]	20[d]	1837	35	235	279	448
Inland (1967)	67-38	W14x30	7.32	1473	121	44	152	22	89	5(2)[d]	65[d]	20[d]	1826[d]	21	261	289	448
Slutter et al. (1971)	71(EPIC)	W12x27	4.57	1143	133	51	127	19	102	11(1)	54[d]	20[d]	1759	21	247[h]		448
Thelen (1972)	72-12(75)	W12x58	7.62	1829	158	76	184	19	114	7(2)	83[d]	20[d]	1858	28	383	426	448
Fisher (1973)	73(RF)	W14x30	7.62	1905	159	76	127[i]	19	140	8(2)	54[d]	20[d]	1749	33	258	270	448
Fisher et. al (1967)	67-11(B1)	W12x27	4.57	1143	140	76	103	19	127	4(2)	42	20[d]	1858	30	253	285	521
	67-11(B2)	W12x27	4.57	1143	140	76	103	19	127	4(2)	42	20[d]	1858	34	253	285	521
Rivera et al. (1968)	68-4(1)	W14x30	6.10	1194	102	38	57	19	76	13(1)	19[d]	20[d]	1873	30	250	278	448
Slutter (1968)	68-5(2)	B16x26	9.14	1156	102	38	57	19	76	14(1)	19[d]	20[d]	1778	23	284	351	448
Slutter (1969a)	69-1	W14x30	6.10	1194	102	38	57	19	76	12(1)	19[d]	20[d]	1847	32	251	267	448
Slutter (1969b)	69-12(4)	W18x45	11.13	1207	102	38	57	19	76	18(1)	19[d]	20[d]	1778	35	257	305	448
Seek et al. (1970)	70-31(A)	W14x30	5.79	1219	102	38	57	19	76	8(1), 5(2)	19, 19	20[d]	1858	23	252	252	521
	70-31(D)	W14x30	5.79	1219	102	38	57	13	76	5(1), 2(9)	22, 22	20[d]	1858	23	252	252	521
	70-31(C)	W18x60	10.82	1829	152	76	143	19	127	16(1), 1(2)	62, 62	20[d]	1858	23	223	259	521
Robinson (1969)	69-2(HR)	W10x21	6.40	1270	157	76	67	19	127	6(1)	24[d]	20[d]	2323	33	259	268	448
Errera (1967)	67-36(CU3)	W12x27	7.32	1588	89	38	57	16	64	12(1), 3(2)	21, 21	20[d]	2323	22	251[g]		448
Hanson (1970)	70-5(C2)	B14x22	7.32	1524	152	51	76	19	102	11(1)	29[d]	20[d]	2323	28	248[d]	248[d]	448
Bethlehem Steel Corporation (1965)	65-19(BS12)	W12x27	4.57	1143	102	33	57	19	76	14(1)	19[d]	20[d]	2323	28	248	285	448
	65-19(BS11)	W12x27	4.57	1143	102	22	44	19	76	14(1)	13[d]	20[d]	2323	28	245	290	448
Errera (1967)	67-36(CU2)	W12x27	7.32	1588	89	38	92	16	64	18(1)	38	20[d]	2323	29	279[g]		448
	67-36(CU1)	W12x27	7.32	1588	89	38	127	16	64	10(1), 2(4)	56, 56	20[d]	2323	30	249[g]		448
Thelen (1972)	72-12(80)	W12x65	7.62	1829	178	76	184	19	114	7(3)	83[d]	20[d]	1852	27	231	259	448
Jones (1975)	75-16	W16x40	9.14	2210	127	76	152	19	114	12(1)	67	18	2323	23	288[f]	336[f]	448

Table I.1(cont'd) Composite Beams with Formed Deck – Test Parameters

Authors	Beam Test	Section	L (m)	b_{eff} (mm)	t_s (mm)	h_r (mm)	w_r (mm)	D (mm)	H_s (mm)	N[a, b]	e_{mid-ht} (mm)	Deck Ga.	w_c (kg/m^3)	f'_c (MPa)	F_{yf} (MPa)[c]	F_{yw} (MPa)[c]	F_u (MPa)
Robinson (1988)	1	W16x36	9.14	2286	142	76	182	19	116	7(1)	3.5(135), 3.5(29)	20[d]	2322.9	21	341	379	448
	2	W16x36	9.14	2286	142	76	153	19	116	8(1)	67	20[d]	2322.9	22	343	377	448
Jayas et al. (1989)	JB-1	W12x35	4.11	1029	152	76	225	19	127	5(2)	46[d]	20[d]	2340.522	27	341	341	448
	JB-2	W12x35	2.04	511	152	76	144	19	127	5(2)	62	20[d]	2399.796	30	341	341	448
	JB-3	W12x35	4.11	1029	152	76	153	19	127	6(2)	46[d]	20[d]	2319.696	24	300	301	448
	JB-4	W12x35	4.11	1029	152	76	153	19	127	6(1)	46[d]	20[d]	2319.696	24	302	313	448
Easterling et al. (1993)	1	W16x31	9.14	2057	152	76	152	19	127	6(1)	105	20	2322.9	33	290	324	447
	2	W16x31	9.14	2057	152	76	152	19	127	6(1)	29	20	2322.9	22	289	313	447
	3	W16x31	9.14	2057	152	76	152	19	127	5(1)	3(105), 2(29)	20	2322.9	16	293	324	447
	4	W16x31	9.14	2057	152	76	152	19	127	5(1)	3(105), 2(29)	20	2322.9	34	301	339	447
Rambo-Roddenberry et al. (2002)	1	W16x31	9.14	2057	127	51	152	19	89	12(1)	105	20	2263.626	34	374	401	461
	2	W16x31	9.14	2057	127	51	152	19	89	12(1)	29	20	2260.422	33	374	401	461
	3	W16x31	9.14	2057	127	51	152	19	89	6(2)	105	20	2297.268	39	374	401	461
Furlong et al. (1975)	TEX-1	W16x50	9.75	2438	159	76	152	19	114	13(2)	67[i]	18	1746.18	21	265	252	448
	TEX-2	W16x50	9.75	2438	159	76	152	19	140	13(2)	67[i]	18	1730.16	26	246	312	448
	TEX-3	W16x50	9.75	2438	159	76	152	19	152	13(2)	67[i]	18	1698.12	26	247	253	448
	TEX-4	W16x50	9.75	2438	159	76	152	19	127	13(2)	67[i]	18	1640.448	24	251	251	448
	TEX-5	W16x50	9.75	2438	159	76	152	19	127	13(2)	67[i]	18	1640.448	24	268	252	448
	TEX-6	W16x50	9.75	2438	159	76	152	19	114	13(2)	67[i]	18	1635.642	25	262	258	448
	TEX-7	W16x50	9.75	2438	159	76	152	19	127	13(2)	67[i]	18	1707.732	27	274	269	448
	TEX-8	W16x50	9.75	2438	159	76	152	19	140	13(2)	67[i]	18	1682.1	28	274	269	448
Allan et al. (1976)	HHR-1-76	W16x45	9.75	2438	140	76	152	19	127	6(2)	8(67), 2(105), 2(29)	18	2306.88	29	256	291	448
	IR-1-76	W16x45	9.75	2438	140	76	184	19	127	6(2)	8(83), 4(129)	18	2306.88	29	255	273	448
	HHR-2-76	W16x45	9.75	2438	140	76	152	19	127	6(2)	8(67), 2(105), 2(29)	18	2322.9	32	254	285	448
	IR-2-76	W16x45	9.75	2438	140	76	184	19	127	6(2)	8(83), 4(129)	18	2306.88	33	257	275	448
	RF-1-76	W16x45	9.75	2438	140	76	152	19	127	6(2)	8(83), 4(129)	18	2322.9	30	254	291	448
	RF-2-76	W16x45	9.75	2438	140	76	152	19	127	6(2)	8(67), 4(105)	18	2290.86	30	254	282	448
Lacap (1975)	175-75	W24x55	10.64	2616	159	76	184	19	127	4(1), 7(2)	129, 129	16	1858.32	28	268	263	448
	174-75	W24x61	10.64	2616	229	76	184	19	178	5(2), 6(3)	129, 83	16	2306.88	29	247	253	448
Lacap (1976)	16-76	W21x44	12.19	2400	140	76	178[i]	19	124	10(2)	80[d]	20[d]	2322.9	24	280	292	448

[a] Total number of studs between the points of zero and maximum moment
[b] C(D) means C ribs with D studs per rib
[c] Static yield point
[d] Estimated
[e] Wet weight
[f] Dynamic yield stress
[g] Averaged, flange and web
[h] Flange stress assumed for both flange and web

Table I.2 Composite Beams with Formed Deck –Results and Theoretical Values

Authors	Beam Test	M_e (kNm)	M_{AISC} (kNm)	M_R (kNm)	M_{RS} (kNm)	$(N/N_f \text{x} 100)_{AISC}$	$(N/N_f \text{x} 100)_R$	$(N/N_f \text{x} 100)_{RS}$	M_e/M_{AISC}	M_e/M_R	M_e/M_{RS}
Grant et al. (1977)	1A1R	826	894	843	876	50.3	34.4	43.0	0.92	0.98	0.94
	1A2	705	773	742	775	51.1	41.5	51.8	0.91	0.95	0.91
	1A3R	788	807	932	907	28.6	54.9	48.4	0.98	0.85	0.87
	1A5R	1026	924	841	875	42.4	24.1	30.2	1.11	1.22	1.17
	1A6R	945	1009	933	912	57.0	37.4	33.0	0.94	1.01	1.04
	1A7	699	823	846	869	36.0	40.9	45.1	0.85	0.83	0.80
	1B1	650	655	744	718	46.0	74.8	66.0	0.99	0.87	0.91
	1B2	682	762	661	689	76.1	35.3	44.1	0.90	1.03	0.99
	1C1	690	685	649	679	28.3	21.6	76.8	1.01	1.06	1.02
	1C2A	711	696	752	732	14.4	20.9	18.4	1.02	0.95	0.97
	1C2B	728	719	798	775	16.1	25.8	22.8	1.01	0.91	0.94
	1C3	783	766	764	746	18.7	18.5	16.4	1.02	1.02	1.05
	1C4	842	823	888	908	18.8	26.4	29.1	1.02	0.95	0.93
	1D1	813	891	840	873	50.6	34.6	43.2	0.91	0.97	0.93
	1D2	698	794	717	744	77.3	42.7	53.3	0.88	0.97	0.94
	1D3	846	918	951	971	30.3	35.3	38.9	0.92	0.89	0.87
	1D4	718	868	866	889	40.3	39.8	43.9	0.83	0.83	0.81
Robinson et al. (1971)	71-17(A1)	196	233	197	219	51.5	35.3	42.1	0.84	0.99	0.89
	71-17(A2)	224	269	230	242	100.0	60.1	71.5	0.83	0.97	0.93
	71-17(A3)	228	262	220	231	96.6	53.8	64.0	0.87	1.04	0.99
	71-17(A4)	227	260	226	237	100.0	61.0	72.6	0.87	1.01	0.96
	71-17(A5)	260	287	242	237	100.0	61.0	72.6	0.90	1.07	1.10
	71-17(B1)	266	264	264	264	100.0	100.0	100.0	1.00	1.00	1.00
	71-17(B2)	212	244	231	242	80.9	65.3	77.7	0.87	0.92	0.87
	71-17(B3)	247	259	256	259	100.0	94.9	100.0	0.95	0.96	0.95
	71-17(B4)	270	260	260	260	100.0	100.0	100.0	1.04	1.04	1.04
Michalski et al. (1970)	70-4	361	354	345	370	62.3	54.5	76.8	1.02	1.05	0.98
Poletto et al. (1970)	70-3(A)	303	353	306	330	100.0	66.5	83.1	0.86	0.99	0.92
	70-3(B)	317	353	306	330	100.0	66.5	83.1	0.90	1.04	0.96
Jones et al. (1966)	66-11(42)	139	125	117	125	31.6	25.0	31.3	1.11	1.19	1.11
	66-11(56)	147	127	119	127	41.9	33.6	42.0	1.17	1.24	1.17
	66-11(W)	382	364	338	353	75.3	51.5	64.4	1.05	1.13	1.08
Slutter et al. (1964)	64-15(H1)	331	288	300	292	61.4	73.3	64.6	1.15	1.10	1.13
	64-15(E1)	323	287	278	270	91.9	80.3	70.8	1.12	1.16	1.20
	64-15(E2)	330	301	294	284	100.0	92.0	81.2	1.10	1.12	1.16
Inland (1967)	67-38	411	383	374	383	71.6	64.6	71.3	1.07	1.10	1.07
Slutter et al. (1971)	71(EPIC)	327	267	297	289	51.7	75.5	68.9	1.23	1.10	1.13
Thelen (1972)	72-12(75)	716	786	787	802	23.9	24.0	26.4	0.91	0.91	0.89
Fisher (1973)	73(RF)	415	475	387	455	96.9	55.5	86.8	0.87	1.07	0.91
Fisher et. al (1967)	67-11(B1)	252	261	264	281	34.5	35.8	44.7	0.97	0.96	0.90
	67-11(B2)	247	268	264	282	37.4	35.8	44.7	0.92	0.93	0.88
Rivera et al. (1968)	68-4(1)	329	349	330	346	69.6	53.4	66.8	0.94	1.00	0.95
Slutter (1968)	68-5(2)	385	383	369	394	57.5	48.5	68.4	1.00	1.04	0.98
Slutter (1969a)	69-1	342	343	322	337	67.9	50.0	62.5	1.00	1.06	1.01
Slutter (1969b)	69-12(4)	598	661	622	647	66.2	46.6	58.1	0.91	0.96	0.92
Seek et al. (1970)	70-31(A)	358	349	342	355	90.8	81.8	100.0	1.03	1.05	1.01
	70-31(D)	343	316	306	318	53.7	44.7	55.6	1.09	1.12	1.08
	70-31(C)	847	769	847	818	43.5	66.1	57.1	1.10	1.00	1.03
Robinson (1969)	69-2(HR)	176	191	187	199	38.7	36.9	43.9	0.92	0.94	0.88
Errera (1967)	67-36(CU3)	270	260	250	262	68.2	56.6	70.7	1.04	1.08	1.03
Hanson (1970)	70-5(C2)	323	316	276	299	94.5	65.1	81.4	1.02	1.17	1.08
Bethlehem Steel Corporation (1965)	65-19(BS12)	300	301	275	290	93.1	64.1	80.2	1.00	1.09	1.03
	65-19(BS11)	310	301	275	290	93.1	64.2	80.2	1.03	1.13	1.07
Errera (1967)	67-36(CU2)	317	303	276	290	80.5	53.5	66.9	1.05	1.15	1.09
	67-36(CU1)	340	287	271	278	100.0	79.4	87.6	1.18	1.26	1.22
Thelen (1972)	72-12(80)	574	638	648[a]	669	41.5	43.6	48.1	0.90	0.89	0.86
Jones (1975)	75-16	548	569	587	599	39.4	44.6	49.2	0.96	0.93	0.91

Table I.2(cont'd) Composite Beams with Formed Deck – Results and Theoretical Values

Authors	Beam Test	M_e (kNm)	M_{AISC} (kNm)	M_R (kNm)	M_{RS} (kNm)	$(N/N_f \times 100)_{AISC}$	$(N/N_f \times 100)_R$	$(N/N_f \times 100)_{RS}$	M_e/M_{AISC}	M_e/M_R	M_e/M_{RS}
Robinson (1998)	1	539	516	516	537	21.0	21.2	24.7	1.04	1.04	1.00
	2	538	537	556	571	24.5	28.4	31.4	1.00	0.97	0.94
Jayas et al. (1989)	JB-1	461	504	450	437	51.7	32.6	28.7	0.91	1.02	1.05
	JB-2	433	466	441	450	43.7	32.6	35.9	0.93	0.98	0.96
	JB-3	402	446	428	415	52.2	44.3	39.1	0.90	0.94	0.97
	JB-4	407	374	381	369	24.0	25.7	22.6	1.09	1.07	1.10
Easterling et al. (1993)	1	412	435	394	427	34.1	24.1	31.9	0.95	1.05	0.97
	2	370	394	356	395	25.6	17.3	26.0	0.94	1.04	0.94
	3	383	357	382	383	16.1	21.1	21.5	1.07	1.00	1.00
	4	411	428	392	417	28.0	20.4	25.5	0.96	1.05	0.99
Rambo-Roddenberry et al. (2002)	1	559	598	575	592	54.4	47.1	51.9	0.93	0.97	0.94
	2	514	594	528	559	53.0	33.2	41.5	0.86	0.97	0.92
	3	510	638	553	568	69.2	40.1	44.1	0.80	0.92	0.90
Furlong et al. (1975)	TEX-1	677	675	745	779	49.0	70.1	81.5	1.00	0.91	0.87
	TEX-2	899	858	775	826	92.3	66.9	82.1	1.05	1.16	1.09
	TEX-3	860	810	733	783	99.3	73.0	89.6	1.06	1.17	1.10
	TEX-4	752	733	734	783	72.4	72.5	89.0	1.03	1.03	0.96
	TEX-5	790	754	754	804	69.3	69.5	85.2	1.05	1.05	0.98
	TEX-6	725	695	752	802	53.3	69.9	85.7	1.04	0.96	0.90
	TEX-7	820	803	776	827	74.7	67.0	82.1	1.02	1.06	0.99
	TEX-8	868	870	778	830	94.7	66.9	82.0	1.00	1.12	1.05
Allan et al. (1976)	HHR-1-76	593	622	561	586	51.4	34.0	40.8	0.95	1.06	1.01
	IR-1-76	645	639	556	581	63.0	36.7	43.4	1.01	1.16	1.11
	HHR-2-76	590	627	556	580	55.4	34.5	41.4	0.94	1.06	1.02
	IR-2-76	639	659	560	585	67.6	36.4	43.1	0.97	1.14	1.09
	RF-1-76	606	625	565	589	53.1	35.9	42.4	0.97	1.07	1.03
	RF-2-76	611	615	561	586	52.4	36.3	42.9	0.99	1.09	1.04
Lacap (1975)	175-75	1072	1066	1017	1039	60.4	49.9	55.0	1.01	1.05	1.03
	174-75	1470	1475	1311	1300	100.0	71.2	69.6	1.00	1.12	1.13
Lacap (1976)	16-76	932	858	810	828	78.8	61.4	67.7	1.09	1.15	1.13

[a] R_n was taken as 0.75 for 3 studs per rib.

Table I.3 Composite Beams with Solid Slabs

Authors	Beam Test	Section	L (m)	b_{eff} (mm)	t_s (mm)	D (mm)	N	w_c (kg/m^3)	f'_c (MPa)	F_{yf} (MPa)	F_{yw} (MPa)	F_u (MPa)	M_e (kNm)	M_{AISC} (kNm)	$(N/N_f \times 100)_{AISC}$	M_e/M_{AISC}
McGarraugh and Baldwin (1971)	B2	W14x30	6.71	914	114	19	6	1794	38	252[b]		448[a]	308.5	326.9	52.1	0.94
	B3	W14x30	6.71	1676	114	19	6	1794	39	246[b]		448[a]	301.7	325.8	53.9	0.93
	B4	W14x30	6.71	1676	114	19	6	1362	34	244[b]		448[a]	282.5	299.3	39.8	0.94
	B5	W14x30	6.71	914	114	19	6	1362	34	248[b]		448[a]	287.0	300.5	39.2	0.96
	B6	W14x30	6.71	914	114	19	12	1794	31	250[b]		448[a]	358.2	361.8	89.2	0.99
Vogel (1971)	PN-1L	W16x45	6.71	1219	127	19	10	2195	26	214[d]	228[d]	448[a]	502.8	499.8	58.5	1.01
	PN-1R	W16x45	6.71	1219	127	19	10	2195	26	214[d]	228[d]	448[a]	498.3	499.8	58.5	1.00
Skinner (1971)	PL-1L	W16x45	6.71	1219	127	19	10	1490	16	207[d]	228[d]	448[a]	436.0	422.4	30.5	1.03
	PL-1R	W16x45	6.71	1219	127	19	10	1490	16	207[d]	228[d]	448[a]	428.6	422.4	30.5	1.01
Baldwin et al. (1965)	LFB7-1	12WF27	6.10	1219	152	22	4	1668	42	257[d]		448[a]	264.4	299.5	52.5	0.88
	NFB4-1	12WF27	6.10	1219	152	13	12	2397	45	258[d]		448[a]	284.8	298.6	51.4	0.95
	LFB5-1	12WF27	6.10	914	152	16	22	1632	21	414[d]		448[a]	515.3	453.4	53.1	1.14
	NFB5-1	12WF27	6.10	914	152	16	38	2403	39	426[d]		448[a]	641.8	588.0	100.0	1.09
	LFB7-2	12WF27	6.10	914	152	22	28	1648	29	414[d]		448[a]	533.3	547.7	100.0	0.97
Proctor (1963)	B1	12WF27	6.10	1219	152	13	16	1730	53	273	317	448[a]	333.6	360.0	62.6	0.93
	B2	12WF27	6.10	1219	152	13	18	1730	56	273	317	448[a]	339.0	376.8	70.4	0.90
Slutter and Driscoll (1965)	B3-T7	12WF27	4.57	1143	102	13	18	2323[a]	25	269	305	461	284.1	292.9	60.3	0.97
	B4-T8S	12WF27	4.57	1143	102	13	18	2323[a]	25	269	305	461	295.3	292.9	60.3	1.01
	B6-T2	12WF27	4.57	1143	102	13	11	2323[a]	25	269	305	461	273.0	261.4	36.9	1.04
	B7-T4	12WF27	4.57	1143	102	13	20	2323[a]	23	258	289	461	288.6	284.9	66.4	1.01
	B8-T9	12WF27	4.57	1143	102	6	16	2323[a]	23	258	289	460	281.5	271.7	53.1	1.04
	B9-T10	12WF27	4.57	1143	102	19	8	2323[a]	23	258	289	523	284.1	278.5	59.7	1.02
	B10-T13	12WF27	4.57	1143	102	13	20	2323[a]	25	252	308	461	293.3	292.9	69.2	1.00
	B11-T13	12WF27	4.57	1143	102	13	20	2323[a]	25	252	308	461	288.8	292.9	69.2	0.99
	B12-T13	12WF27	4.57	1143	102	13	20	2323[a]	25	252	308	461	296.7	292.9	69.2	1.01
Barnard and Johnson (1965)	SS1	BSB8x5¼x20	3.66	254	127	13	22	2323[a]	51	293	322	448[a]	188.8	204.6	100.0	0.92
	SS2	BSB8x5¼x20	3.66	305	127	13	22	2323[a]	53	293	322	448[a]	200.0	216.6	100.0	0.92
	SS3	BSB8x5¼x20	3.66	406	127	13	22	2323[a]	39	293	322	448[a]	217.0	215.2	100.0	1.01
	SS4	BSB8x5¼x20	3.66	457	127	13	22	2323[a]	32	293	322	448[a]	191.5	212.1	100.0	0.90
	SS5	BSB8x5¼x20	3.66	610	127	13	22	2323[a]	49	293	322	448[a]	220.4	238.5	100.0	0.92
	SS6	BSB8x5¼x20	3.66	616	127	13	22	2323[a]	40	293	322	448[a]	232.8	232.7	100.0	1.00
Lew (1970)[j]	3G1	-	-	-	-	-	-	-	-	-	-	-	217.0	231.6	-	0.94
	4G1	-	-	-	-	-	-	-	-	-	-	-	214.1	243.0	-	0.88
	5G1	-	-	-	-	-	-	-	-	-	-	-	202.3	240.7	-	0.84
	3U1	-	-	-	-	-	-	-	-	-	-	-	194.9	196.6	-	0.99
	4U1	-	-	-	-	-	-	-	-	-	-	-	207.9	206.8	-	1.01
	5U1	-	-	-	-	-	-	-	-	-	-	-	198.9	211.3	-	0.94
Culver and Coston (1961)	B1-T1	8WF17	3.05	610	76	13	24	2323[a]	38	254[i]		448[a]	131.5	129.0	100.0	1.02
	B2-T2	8WF17	3.05	610	76	13	14	2323[a]	38	254[i]		448[a]	130.6	128.0	96.8	1.02
	B3-T3	8WF17	3.05	610	76	13	10	2323[a]	38	254[i]		448[a]	121.1	118.1	69.2	1.03

Table I.3(cont'd) Composite Beams with Solid Slabs

Authors	Beam Test	Section	L (m)	b_{eff} (mm)	t_s (mm)	D (mm)	N	w_c (kg/m^3)	f'$_c$ (MPa)	F_{yf} (MPa)	F_{yw} (MPa)	F_u (MPa)	M_e (kNm)	M_{AISC} (kNm)	$(N/N_f \times 100)_{AISC}$	M_e/M_{AISC}
Yam and Chapmann (1969)	E2	BSB12x6x44	5.49	1219	152	19	42	2323^a	29	211^b		448^a	525	488	100.0	1.08
	E3	BSB12x6x44	5.49	1219	152	19	38	2323^a	34	212^b		448^a	549	498	100.0	1.10
	E4	BSB12x6x44	5.49	1219	152	19	34	2323^a	25	245^b		448^a	549	544	100.0	1.01
	E5	BSB12x6x44	5.49	1219	152	19	28	2323^a	31	245^b		448^a	635	562	100.0	1.13
	E6	BSB12x6x44	5.49	1219	152	19	22	2323^a	41	229^b		448^a	574	542	100.0	1.06
	E7	BSB12x6x44	5.49	1219	152	19	16	2323^a	41	223^b		448^a	519	530	100.0	0.98
	E8	BSB12x6x44	5.49	1219	152	19	22	2323^a	39	210^b		448^a	598	498	100.0	1.20
	E9	BSB12x6x44	5.49	1219	152	19	22	2323^a	39	226^b		448^a	549	534	100.0	1.03
	E10	BSB12x6x44	5.49	1219	152	13	50	2323^a	33	236^b		448^a	561	546	100.0	1.03
	E11	BSB12x6x44	5.49	1219	152	13	50	2323^a	50	237^b		448^a	633	568	100.0	1.11
Toprac (1965)	11	W12(2$^3_4$/5$^1_4$)84	4.57	1143	102	13	24	2323^a	26	229	243	448^a	295	266	100.0	1.11
	12	W12(2$^3_4$/5$^1_4$)84	4.57	1143	102	13	16	2323^a	26	294^e/229^f	243	448^a	281	250	75.8	1.13
	13	W12484	4.57	1143	102	13	7	2323^a	26	294^e/229^f	243	448^a	230	185	32.4	1.24
	14a	W12(2$^3_4$/5$^1_4$)84	4.57	1143	102	13	7	2323^a	26	294^e/229^f	243	448^a	219	205	33.2	1.07
	14b	W12(2$^3_4$/5$^1_4$)84	4.57	1143	102	13	7	2323^a	26	294^e/229^f	243	448^a	234	205	33.2	1.14
	15	W12(2$^3_4$/5$^1_4$)84	4.57	1143	102	13	4	2323^a	26	294^e/229^f	243	448^a	189	175	18.9	1.08
	21	W12484	4.57	1143	102	13	24	2323^a	26	229.0	243	448^a	241	239	100.0	1.01
	22a	W12484	4.57	1143	102	13	26	2323^a	26	229^e/763^f	243	448^a	494	460	77.8	1.07
	22b	W12484	4.57	1143	102	13	26	2323^a	26	229^e/763^f	243	448^a	494	460	77.8	1.07
	23	W12484	4.57	1143	102	13	36	2323^a	26	229^e/365^f	243	448^a	331	302	100.0	1.10
	32	W12484	4.57	1143	102	13	28	2323^a	26	229^e/763^f	243/353^g	448^a	537	478	81.5	1.12
	33a	W12484	4.57	1143	102	13	48	2323^a	26	229^e/365^f	243/731^h	448^a	490	441	100.0	1.11
	33b	W12484	4.57	1143	102	13	48	2323^a	26	229^e/365^f	243/731^h	448^a	499	441	100.0	1.13
	34	W12484	4.57	1143	102	13	28	2323^a	26	229^e/365^f	243/353^g	448^a	343	317	100.0	1.08
	35	W12484	4.57	1143	102	13	48	2323^a	26	229^e/763^f	243/1034^h	448^a	707	679	93.1	1.04

[a] Estimated

[b] Weighted average of flange and web yield stresses

[c] Average reported strength

[d] Average static yield point

[e] Top flange

[f] Bottom flange

[g] (Yield strength of top 211 mm of web)/(Yield strength of bottom 68 mm of web)

[h] (Yield strength of top 95 mm of web)/(Yield strength of bottom 184 mm of web)

[i] Flange yield stress is also taken as the web yield stress

[j] Computed values were taken from Galambos et al. (1976)

Table I.4 Statistical characteristics of shear connection

DESCRIPTION	$\left[\frac{Q_e}{Q_p}\right]_m$	V_P	$\frac{(Q_u)_m}{(Q_u)_n}$	V_{Q_u}
1999 AISCS, ribbed slabs (230 tests)	0.78	0.28	0.88	0.34
1999 AISCS, all tests (254 tests)	0.80	0.28	0.90	0.34
Either method, solid slabs (24 tests)	1.01	0.14	1.14	0.24
RS, ribbed slabs (175 tests)	0.84	0.23	0.95	0.30
RS, all tests (199 tests)	0.86	0.22	0.97	0.30
R, ribbed slabs (202 tests)	1.02	0.15	1.17	0.18
R, all tests (226 tests)	1.02	0.16	1.15	0.18*

*Assuming $A_{sc}F_u$ governs for solid slab tests

Table I.5 Summary of Statistical Characteristics of Beam Tests

DESCRIPTION	$\left[\frac{M_e}{M_p}\right]_m$	V_P
Solid Slabs – P.N.A. in web	1.03	0.06
Solid Slabs – P.N.A. in flange	1.02	0.08
Solid Slabs – P.N.A. in concrete slab	1.04	0.07
Ribbed Slabs – P.N.A. in web (1999 AISCS)	0.99	0.09
Ribbed Slabs – P.N.A. in flange (1999 AISCS)	0.98	0.08
Ribbed Slabs – P.N.A. in concrete slab (1999 AISCS)	0.97	0.11
Ribbed Slabs – P.N.A. in web (R)	1.01	0.09
Ribbed Slabs – P.N.A. in flange (R)	1.05	0.09
Ribbed Slabs – P.N.A. in concrete slab (R)	1.02	0.02
Ribbed Slabs – P.N.A. in web (RS)	1.00	0.08
Ribbed Slabs – P.N.A. in flange (RS)	1.00	0.10
Ribbed Slabs – P.N.A. in concrete slab (RS)	1.00	0.04
Both Types of Slabs – P.N.A. in web (1999 AISCS)	1.00	0.08
Both Types of Slabs – P.N.A. in flange (1999 AISCS)	0.99	0.08
Both Types of Slabs – P.N.A. in concrete slab (1999 AISCS)	1.02	0.09
Both Types of Slabs – P.N.A. in web (R)	1.03	0.09
Both Types of Slabs – P.N.A. in flange (R)	1.05	0.08
Both Types of Slabs – P.N.A. in concrete slab (R)	1.03	0.06
Both Types of Slabs – P.N.A. in web (RS)	1.01	0.08
Both Types of Slabs – P.N.A. in flange (RS)	1.00	0.09
Both Types of Slabs – P.N.A. in concrete slab (RS)	1.04	0.07

If ***the P.N.A. is in the concrete slab***, $C = A_s F_y < 0.85 f'_c b_{eff} t_s, \Sigma Q_u$:

$$\left(M_u\right)_n = A_s F_y \left[t_s + \frac{d}{2} - \frac{A_s F_y}{1.7 f'_c b_{eff}} \right] \qquad \text{(Eq. I.11)}$$

and,

$$\left(M_u\right)_m = \left[\frac{M_e}{M_p}\right]_m \left\{ A_s \left(F_y\right)_m \left[t_s + \frac{d}{2} - \frac{A_s \left(F_y\right)_m}{1.7 \left(f'_c\right)_m b_{eff}} \right] \right\} \qquad \text{(Eq. I.12)}$$

Introducing $\xi_1 = \dfrac{A_s F_y}{0.85 b_{eff} t_s f'_c}$, and $\xi_2 = \dfrac{d}{t_s}$, we can write:

$$\frac{R_m}{R_n} = \frac{\left(M_u\right)_m}{\left(M_u\right)_n} = \left[\frac{M_e}{M_p}\right] \left\{ \frac{ \dfrac{\left(F_y\right)_m}{\left(F_y\right)_n} \left[1 + \dfrac{\xi_2}{2} - \dfrac{\left(F_y\right)_m \left(f'_c\right)_n}{\left(F_y\right)_n \left(f'_c\right)_m} \dfrac{\xi_1}{2} \right] }{ 1 + \dfrac{\xi_2 - \xi_1}{2} } \right\} \qquad \text{(Eq. I.13)}$$

The parameter ξ_1 was investigated in its theoretical range from zero to one. The value of ξ_2 is not theoretically constrained, however, in this study it was investigated in the range of values from one to seven, as those two values were determined to be the limits of practical applications. Standard deviation associated with materials was determined as shown by Eq. I.14.

$$\sigma_M = \sqrt{ \left(\frac{\partial M_u}{\partial F_y} \right)_m^2 \sigma_{F_y}^2 + \left(\frac{\partial M_u}{\partial f'_c} \right)_m^2 \sigma_{f'_c}^2 } \qquad \text{(Eq. I.14)}$$

Finally, V_M was calculated using Eq. I.15.

$$V_M = \frac{ \sqrt{ \left[1 + \dfrac{\xi_2}{2} - \dfrac{\left(F_y\right)_m \left(f'_c\right)_n}{\left(F_y\right)_n \left(f'_c\right)_m} \dfrac{\xi_1}{2} \right]^2 \sigma_{F_y}^2 + \left[\dfrac{\xi_1}{2} \dfrac{\left(F_y\right)_m^2 \left(f'_c\right)_n^2}{\left(F_y\right)_n^2 \left(f'_c\right)_m^2} \right]^2 \sigma_{f'_c}^2 } }{ \dfrac{\left(F_y\right)_m}{\left(F_y\right)_n} \left[1 + \xi_2 - \dfrac{\left(F_y\right)_m \left(f'_c\right)_n}{\left(F_y\right)_n \left(f'_c\right)_m} \dfrac{\xi_1}{2} \right] } \qquad \text{(Eq. I.15)}$$

It was observed that the computed value of resistance factor generally increases with the increasing value of ξ_1, and decreases with the increasing value of ξ_2. A typical set of calculations is shown in Table I.6. The values shown correspond to 1999 AISCS, and statistics generated based on ribbed-slab test results.

Table I.6 Resistance Factor Calculations for $0 \le \xi_1 \le 1$ and $\xi_2 = 5.0$

ξ_1	ξ_2	$(M_u)_m/(M_u)_n$	V_M	V_R	Φ	$\Phi_{average}$
0.0	**5.0**	1.08	0.11	0.13	0.88	**0.89**
0.1	**5.0**	1.08	0.11	0.13	0.88	
0.2	**5.0**	1.09	0.11	0.12	0.88	
0.3	**5.0**	1.09	0.11	0.12	0.89	
0.4	**5.0**	1.09	0.10	0.12	0.89	
0.5	**5.0**	1.10	0.10	0.12	0.89	
0.6	**5.0**	1.10	0.10	0.12	0.89	
0.7	**5.0**	1.11	0.10	0.12	0.90	
0.8	**5.0**	1.11	0.10	0.12	0.90	
0.9	**5.0**	1.12	0.10	0.12	0.90	
1.0	**5.0**	1.12	0.10	0.12	0.90	

To determine the effect of shear connectors on the resistance factor, it was first necessary to define the required statistical parameters pertaining to shear connector strength. The following illustrates how the values of $(Q_u)_m/(Q_u)_n$ and V_{Q_u} were calculated for solid slabs, and what the required adjustments are for Eqs. I.13 and I.15. All the computed values are summarized in Table I.4. In the analysis, it was assumed that $(w_c)_m=1.0(w_c)_n$. Considering Eq. I.2 we write:

$$(Q_u)_n = \min\left\{ \begin{matrix} 0.10A_{sc}w_c^{0.75}\left(f_c^{'}\right)^{0.75} \\ A_{sc}F_u \end{matrix} \right\} \qquad \text{(Eq. I.16)}$$

If concrete strength governs:

$$(Q_u)_m = \left(\frac{Q_e}{Q_p}\right)_m \left[0.10A_{sc}w_c^{1.5}\left(f_c^{'}\right)_m^{0.75}\right] \qquad \text{(Eq. I.17)}$$

and by combining the Eqs. I.16 and I.17, we write:

$$\frac{(Q_u)_m}{(Q_u)_n} = \left(\frac{Q_e}{Q_p}\right)_m \left[\frac{\left(f_c^{'}\right)_m}{f_c^{'}}\right]^{0.75} \qquad \text{(Eq. I.18)}$$

Finally, the value of V_{Q_u} is computed as follows:

$$V_{Q_u} = \sqrt{V_P^2 + V_{(f_c')^{0.75}}^2} \qquad \text{(Eq. I.19)}$$

where:

$$V_{(f_c')^{0.75}}^2 = \frac{\left(\frac{\partial Q_u}{\partial f_c'}\right)^2 \sigma_{f_c'}^2}{(Q_u)_m^2} \qquad \text{(Eq. I.20)}$$

In a similar manner, the values of $(Q_u)_m/(Q_u)_n$ and V_{Q_u} are obtained considering the right side of Eqs. I.2 and I.5, and these are also given in Table I.4. The right side of Eq. I.2 was observed to govern in only 2% of considered ribbed-slab tests, and in 36% of considered solid slab tests. The situation is quite different with respect to Eq. I.6 and the RS-method, where the right side governs in 93% of considered ribbed-slab tests. However, the values of $(Q_u)_m/(Q_u)_n$ and V_{Q_u} determined based on the tensile strength of a stud were not used in the analysis for both methods, as they are somewhat less conservative that those based on concrete strength. The exception to this is the model by Roddenberry et al. (2002a) where the concrete strength as a variable was eliminated. When this method is analyzed, and a combination of solid-slab and ribbed-slab tests considered, two parameters were determined based on $A_{sc}F_u$ governing in solid slabs. This was done to make the statistical characteristics compatible for combining and it is justified considering the relative size of samples and the fact that $A_{sc}F_u$ governed in 64% of solid slab tests. To consider their effect on the resistance factor, the Eq. I.11 was rewritten as follows:

$$(M_u)_n = A_s F_y \left[t_s + \frac{d}{2} - \frac{\Sigma Q_u}{1.7 f_c' b_{eff}} \right] \qquad \text{(Eq. I.21)}$$

Following a similar analysis as given above, the expressions for $(R)_m/(R)_n$ and V_M are given as Eq. I.22 and I.23, respectively. Typical calculations of the resistance factor are similar to those presented in Table I.6.

$$\frac{R_m}{R_n} = \frac{(M_u)_m}{(M_u)_n} = \left[\frac{M_e}{M_p}\right]\left\{\frac{\frac{(F_y)_m}{(F_y)_n}\left[1 + \frac{\xi_2}{2} - \frac{(Q_u)_m (f_c')_n}{(Q_u)_n (f_c')_m}\frac{\xi_1}{2}\right]}{1 + \frac{\xi_2 - \xi_1}{2}}\right\} \qquad \text{(Eq. I.22)}$$

$$V_M = \frac{\sqrt{\left[\frac{(F_y)_m (f_c')_n}{2(F_y)_n (f_c')_m}\xi_1\right]^2 \sigma_{Q_u}^2 + \left[1 + \frac{\xi_2}{2} - \frac{(Q_u)_m (f_c')_n}{(Q_u)_n (f_c')_m}\frac{\xi_1}{2}\right]^2 \sigma_{F_y}^2 + \left[\frac{\xi_1}{2}\frac{(F_y)_m (f_c')_n (Q_u)_m}{(F_y)_n (f_c')_m (Q_u)_n}\right]^2 \sigma_{f_c'}^2}}{\frac{(F_y)_m}{(F_y)_n}\left[1 + \xi_2 - \frac{(Q_u)_m (f_c')_n}{(Q_u)_n (f_c')_m}\frac{\xi_1}{2}\right]} \qquad \text{(Eq. I.23)}$$

Next, if the P.N.A. is in the steel, then either the slab or shear connection governs, i.e., $C = \min\{0.85 f_c' b_{eff} t_s, \Sigma Q_u\} < A_s F_y$. Within the considered pool of composite beam tests shear connection, rather than the slab, was governing in all the cases where the P.N.A. is the steel. However, both cases are considered in the reliability analysis. When ***<u>the P.N.A. is in the steel flange</u>***, then $A_s F_y - C < 2F_y b_f t_f$, and:

$$M_u = C\left(t_s - \frac{C}{1.7 f_c' b_e}\right) + A_s F_y\left(\frac{d}{2}\right) - (A_s F_y - C)\left(\frac{A_s F_y - C}{4F_y b_f}\right) \qquad \text{(Eq. I.24)}$$

Statistical parameters R_m/R_n and V_M were derived with similar methodology used to obtain Eqs. (I.13,15, 22, 23). The two parameters corresponding to the condition in which concrete slab governs are shown as Eqs. I.25 and I.26.

$$\frac{R_m}{R_n} = \left[\frac{M_e}{M_p}\right]\left\{\frac{\frac{(f_c')_m}{(f_c')_n}\xi_1 t_s + \frac{(F_y)_m}{(F_y)_n} d - \frac{A_s\left[1 - \xi_1 \frac{(f_c')_m (F_y)_m}{(F_y)_n (f_c')_n}\right]^2}{4b_f}}{\xi_1 t_s + d - \frac{A_s (1-\xi_1)^2}{4b_f}}\right\} \qquad \text{(Eq. I.25)}$$

$$V_M = \frac{\sqrt{\left[d - \frac{A_s}{2b_f} + \frac{(F_y)_n^2 (f_c^{'})_m^2 A_s \xi_1^2}{4b_f (f_c^{'})_n^2 (F_y)_m^2}\right]^2 \sigma_{F_y}^2 + \left[t_s \xi_1 + \frac{A_s \xi_1}{2b_f} - \frac{(F_y)_n^2 (f_c^{'})_m^2 A_s \xi_1^2}{b_f (f_c^{'})_n^2 (F_y)_m^2}\right] \sigma_{f_c^{'}}^2}}{\frac{(f_c^{'})_m}{(f_c^{'})_n} \xi_1 t_s + \frac{(F_y)_m}{(F_y)_n} d - \frac{A_s \left[1 - \xi_1 \frac{(f_c^{'})_m (F_y)_m}{(F_y)_n (f_c^{'})_n}\right]^2}{2b_f}} \qquad \text{(Eq. I.26)}$$

To investigate the resistance factor, minimum, average, and maximum values of (A_s/b_f), d, t_s, and ξ_1 were selected. The values of t_s ranged between three and 8 in. with the stipulation that the d/t_s ratio does not exceed 7, nor does it drop below one. The selected values of (A_s/b_f) and d reflect minimum, average, and maximum values chosen from 318 H-shaped members in AISC Manual of Steel Construction (AISC 2001). Because the P.N.A. is in the flange the theoretical range of parameter ξ_1, and therefore its investigated range, was subject to that shown in Eq. I.27.

$$1 - \frac{2b_f t_f}{A_s} \le \xi_1 \le 1 \qquad \text{(Eq. I.27)}$$

The lower boundary of the parameter ranges between 0.18 and 0.42. Table I.7 shows the selected shapes and typical calculated values of the resistance factor. The presented case is based on considering all tests, regardless of slab type. Some of the investigated cases clearly represent extreme situations, not necessarily often used in practical situations. However, they were investigated to cover the widest range of theoretical solutions possible.

When the P.N.A. is in the flange, it is harder to discern the general trend under which the resistance factor is increasing or decreasing. The reason for this is the number and complexity of statistical parameters affecting its magnitude. However, the commonly made observation was that the smaller resistance factors are generally associated with deeper beams, especially those associated with a comparatively smaller slab thickness.

When the P.N.A. is in the flange, consideration of the effect of shear connection strength is computationally rather complex. New parameters had to be introduced and larger number of calculations performed, and Eqs. I.25 and I.26 are rewritten as Eqs. I.28 and I.29.

Table I.7 Typical ϕ for Composite Beams with P.N.A. in Flange and 1999 AISC

SHAPE*	ξ_1	A_s/b_f	d/2	t_s	d	$(M_u)_m/(M_u)_n$	V_M	V_R	Φ	$\Phi_{average}$
W1100x449 (W44x335)	**0.424**	**156**	**559**	**160**	**1118**	1.08	0.099	0.14	0.86	**0.87**
W920x1188 (W36x798)	**0.343**	**332**	**533**	**152**	**1067**	1.09	0.095	0.14	0.87	
W760x185 (W30x124)	**0.465**	**88**	**384**	**152**	**767**	1.08	0.099	0.14	0.86	
W690x240 (W27x161)	**0.365**	**86**	**351**	**152**	**701**	1.08	0.101	0.14	0.86	
W530x82 (W21x55)	**0.470**	**50**	**264**	**152**	**528**	1.08	0.099	0.14	0.86	
W530x74 (W21x50)	**0.525**	**57**	**264**	**152**	**528**	1.08	0.098	0.14	0.86	
W360x1086 (W14x730)	**0.182**	**305**	**284**	**152**	**569**	1.09	0.096	0.14	0.87	
M310x14.9 (M12x10)	**0.603**	**23**	**152**	**152**	**305**	1.09	0.093	0.13	0.87	
M100x8.9 (M4x6)	**0.305**	**12**	**48**	**97**	**97**	1.09	0.095	0.14	0.87	
S75x11.2 (S3x7.5)	**0.407**	**22**	**38**	**76**	**76**	1.11	0.087	0.13	0.89	
S75x8.5 (S3x5.7)	**0.270**	**18**	**38**	**76**	**76**	1.10	0.094	0.14	0.88	

*Parenthesis contain the shape designations expressed in English units.

$$\frac{R_m}{R_n} = \left[\frac{M_e}{M_p}\right]\left\{\frac{\dfrac{(Q_u)_m}{(Q_u)_n}\xi_4\left[t_s - \dfrac{(Q_u)_m (f'_c)_n \xi_3 t_s}{2(Q_u)_n (f'_c)_m}\right] + \dfrac{d(F_y)_m}{2(F_y)_n} - \dfrac{A_s\left[1 - \dfrac{(F_y)_n (Q_u)_m}{(F_y)_m (Q_u)_n}\xi_4\right]^2}{4b_f}}{\xi_4\left(t_s - \dfrac{\xi_3 t_s}{2}\right) + \dfrac{d}{2} - \dfrac{A_s(1-\xi_4)^2}{4b_f}}\right\} \quad \text{(Eq. I.28)}$$

$$V_M = \frac{\sqrt{\begin{aligned}&\left[t_s\xi_3 - \frac{(Q_u)_m (f'_c)_n}{(Q_u)_n (f'_c)_m}\xi_3\xi_4 - \frac{A_s\xi_4}{2b_f} + \frac{(Q_u)_m (F_y)_n A_s \xi_4^2}{2b_f (Q_u)_n (F_y)_m}\right]^2 \sigma_{Q_u}^2 + \left[\frac{(Q_u)_m^2 (f'_c)_n^2}{2(Q_u)_n^2 (f'_c)_m^2}\xi_3\xi_4 t_s\right]^2 \sigma_{f'_c}^2 \\ &+ \left[\frac{d}{2} - \frac{A_s}{4b_f} + \frac{(Q_u)_m^2 (F_y)_n^2}{(Q_u)_n^2 (F_y)_m^2}\frac{A_s}{4b_f}\xi_4^2\right]^2 \sigma_{F_y}^2\end{aligned}}}{\dfrac{(Q_u)_m}{(Q_u)_n}\xi_4\left[t_s - \dfrac{(Q_u)_m (f'_c)_n \xi_3 t_s}{2(Q_u)_n (f'_c)_m}\right] + \dfrac{d(F_y)_m}{2(F_y)_n} - \dfrac{A_s\left[1 - \dfrac{(F_y)_n (Q_u)_m}{(F_y)_m (Q_u)_n}\xi_4\right]^2}{4b_f}} \quad \text{(Eq. I.29)}$$

where:

$$\xi_3 = \frac{\Sigma Q_u}{0.85 f'_c b_{eff} t_s}, \text{ or } \xi_3 = \xi_4\xi_1, \text{ and } \xi_4 = \frac{\Sigma Q_u}{A_s F_y}$$

A method of analysis similar to that when the concrete slab governs was used. The parameter ξ_4 was subject to the same stipulations expressed for ξ_1 in Eq. I.26. In the analysis, ξ_3 was investigated for ξ_1 = 0.25, 0.63, and 1.0. Consideration for other variables, such as d, and t_s is the same as was done when the strength of slab governs. As a general trend, it was observed that when P.N.A. is in flange and shear connection governs, lower values of the resistance are typically associated with smaller beam sections. Typical calculations are shown in Table I.8. The case shown is one in which $\xi_1 = 0.63$, $\xi_4 = 1\text{-}2b_f t_f/A_s$, R-method, and ribbed-slab tests were used.

Finally, the cases where the P.N.A. is in the web were investigated. A characteristic of this condition is a low degree of shear connection, generally below 40%. In fact, the average degree of shear connection for all beams within the considered test database in which P.N.A. is in the web was 30% when computed by 1999 AISC, 34% when computed using RS-method, and 33% if R-method is used. In all such beam tests considered, shear connections was the limiting component, rather than the concrete slab. However, both cases were considered in the analysis. If ***the P.N.A. is in the steel web***, then $A_s F_y - C > 2F_y b_f t_f$, and M_u is written as shown in Eq. I.30. The statistical parameters R_m/R_n and V_M are shown as Eqs. I.31 and I.32, respectively.

Table I.8 Typical ϕ with P.N.A. in Flange, Shear Conn. Governing and RS-method Used

SHAPE*	ξ_3	ξ_4	t_s	$(M_u)_m/(M_u)_n$	V_M	V_R	Φ	$\Phi_{average}$
W1100x449 (W44x335)	**0.265**	**0.424**	**160**	1.14	0.10	0.14	0.91	
W920x1188 (W36x798)	**0.214**	**0.343**	**152**	1.15	0.09	0.14	0.91	
W760x185 (W30x124)	**0.291**	**0.465**	**110**	1.14	0.10	0.14	0.91	
W690x240 (W27x161)	**0.228**	**0.365**	**100**	1.14	0.10	0.14	0.91	
W530x82 (W21x55)	**0.294**	**0.470**	**76**	1.14	0.10	0.14	0.91	
W530x74 (W21x50)	**0.328**	**0.525**	**76**	1.15	0.09	0.14	0.91	**0.92**
W360x1086 (W14x730)	**0.114**	**0.182**	**81**	1.15	0.09	0.14	0.91	
M310x14.9 (M12x10)	**0.377**	**0.603**	**76**	1.15	0.09	0.14	0.92	
M100x8.9 (M4x6)	**0.191**	**0.305**	**76**	1.16	0.08	0.13	0.94	
S75x11.2 (S3x7.5)	**0.254**	**0.407**	**76**	1.18	0.08	0.13	0.95	
S75x8.5 (S3x5.7)	**0.169**	**0.270**	**76**	1.17	0.08	0.13	0.94	

*Parenthesis contain the shape designations expressed in English units.

$$M_u = C\left(t_s - \frac{C}{1.7f'_c b_{eff}}\right) + A_s F_y\left(\frac{d}{2}\right) - (t_f)^2 b_f F_y - \left(A_s F_y - C - 2t_f b_f F_y\right)\left(\frac{A_s F_y - C}{4F_y t_w} - \frac{b_f t_f}{2t_w} + t_f\right) \quad \text{(Eq. I.30)}$$

$$\frac{R_m}{R_n} = \left[\frac{M_e}{M_p}\right]\left\{\frac{0.5t_s\xi_5\frac{(f'_c)_m}{(f'_c)_n} + \frac{(F_y)_m}{(F_y)_n}\frac{d}{2} - \frac{(F_y)_m}{(F_y)_n}\xi_6 t_f - \left[\frac{(F_y)_m}{(F_y)_n}(1-2\xi_6) - \xi_5\frac{(f'_c)_m}{(f'_c)_n}\right]\left[t_f + \frac{1 - \frac{(F_y)_n}{(F_y)_m}\frac{(f'_c)_m}{(f'_c)_n}\xi_5 - 2\xi_6}{4A_s t_w}\right]}{0.5t_s\xi_5 + \frac{d}{2} - \xi_6 t_f - \frac{(1-2\xi_6-\xi_5)^2}{4A_s t_w}}\right\} \quad \text{(Eq. I.31)}$$

where:

$$\left(\xi_5 = \frac{1}{\xi_1}\right) \le 1 - \frac{2}{\xi_6}, \text{ and } \xi_6 = \frac{b_f t_f}{A_s}$$

Investigated H-shaped sections were taken out of AISC Manual of Steel Construction. The 14 beam shapes considered were used to form 756 composite sections that correspond to the minimum, average, and the maximum theoretical values of ξ_5, ξ_6, b_f, t_f, t_w, A_s, t_s, d, and $\xi_6 t_f$. Typical calculations of the resistance factor when the P.N.A. is in the web are shown in Table I.9. The case shown is when $\xi_5 = 0.5-(b_f t_f/A_s)$. The magnitude of resistance factor is dependant on many variables, so a general trend is hard to observe. However, lower values of resistance factors were generally found to correspond to lower values of ξ_5.

$$V_M = \frac{\sqrt{\left[0.5\xi_5 t_s - \frac{A_s\xi_5}{2t_w} - t_f\xi_5 + \left(\frac{b_f t_f}{t_w}\right)\xi_5 + \frac{(F_y)_n}{(F_y)_m}\frac{(f'_c)_m}{(f'_c)_n}\frac{A_s\xi_5^2}{2t_w}\right]^2\sigma^2_{f'_c} + \left[\frac{d}{2} - 3t_f\xi_6 + t_f - \frac{b_f t_f}{t_w} + \frac{A_s}{4t_w} - \frac{(F_y)^2_n}{(F_y)^2_m}\frac{(f'_c)^2_m}{(f'_c)^2_n}\frac{A_s}{4t_w}\xi_5^2 + \frac{A_s\xi_6^2}{t_w}\right]^2\sigma^2_{F_y}}}{0.5t_s\xi_5\frac{(f'_c)_m}{(f'_c)_n} + \frac{(F_y)_m}{(F_y)_n}\frac{d}{2} - \frac{(F_y)_m}{(F_y)_n}\xi_6 t_f - \left[\frac{(F_y)_m}{(F_y)_n}(1-2\xi_6) - \xi_5\frac{(f'_c)_m}{(f'_c)_n}\right]\left[t_f + \frac{1 - \frac{(F_y)_n}{(F_y)_m}\frac{(f'_c)_m}{(f'_c)_n}\xi_5 - 2\xi_6}{4A_s t_w}\right]} \quad \text{(Eq. I.32)}$$

Table I.9 Typical ϕ for Solid Slab Beams, P.N.A. in Web, and Slab Strength Governing

SHAPE*	ξ_5	ξ_6	t_s	V_P	V_F	$(M_u)_m/(M_u)_n$	V_M	V_R	Φ	$\Phi_{average}$
W1100x449 (W44X335)	**0.212**	**0.288**	**160**	0.06	0.05	1.13	0.13	0.16	0.87	**0.88**
W1000x321 (W40X215)	**0.196**	**0.304**	**142**	0.06	0.05	1.13	0.13	0.15	0.88	
W920x223 (W36X150)	**0.245**	**0.304**	**142**	0.06	0.05	1.13	0.14	0.17	0.86	
W690x240 (W27X161)	**0.182**	**0.318**	**102**	0.06	0.05	1.13	0.13	0.15	0.88	
W530x82 (W21X55)	**0.235**	**0.265**	**102**	0.06	0.05	1.13	0.13	0.16	0.88	
W460x213 (W18X143)	**0.149**	**0.351**	**102**	0.06	0.05	1.13	0.13	0.15	0.88	
W360x1202 (W14X808)	**0.098**	**0.402**	**102**	0.06	0.05	1.13	0.13	0.16	0.87	
W360x1086 (W14X730)	**0.091**	**0.409**	**102**	0.06	0.05	1.13	0.13	0.15	0.87	
W250x149 (W10X100)	**0.108**	**0.392**	**102**	0.06	0.05	1.13	0.12	0.15	0.89	
M310x14.9 (M12X10)	**0.302**	**0.198**	**102**	0.06	0.05	1.14	0.13	0.15	0.89	
M150x5.5 (M6X3.7)	**0.263**	**0.237**	**102**	0.06	0.05	1.14	0.12	0.14	0.90	
S310x74 (S12X50)	**0.253**	**0.247**	**102**	0.06	0.05	1.14	0.13	0.16	0.88	
S75x11.2 (S3X7.5)	**0.203**	**0.297**	**76**	0.06	0.05	1.15	0.12	0.14	0.91	
S75x8.5 (S3X5.7)	**0.135**	**0.365**	**76**	0.06	0.05	1.14	0.11	0.14	0.90	

*Parenthesis contain the shape designations in English units.

Lastly, the effect of shear connection was studied for beams with the P.N.A. in the web. The statistical parameters R_m/R_n and V_M are shown as Eqs. I.33 and I.34. The parameters ψ_1 through ψ_4 were introduced for clarity and to simplify the analysis. Others were previously defined. The methodology for selecting the shapes and other parameters for the analyses was the same as used for the case of concrete slab governs. Table I.10 illustrates typical calculations of the resistance factors, where $\xi_3 = \xi_4$. Generally, the effect of various geometric parameters on the resistance factor is hardly discernable when the P.N.A. is in the web, due to the fact that calculated components contain many independent variables, each affecting the resistance factor. However, as will be shown later, for both when slab and shear connection govern, the resistance factor was found to be significantly lower than when the P.N.A. is in either the flange or the concrete slab.

$$\frac{R_m}{R_n} = \left[\frac{M_e}{M_p}\right]\left\{\frac{\xi_4 \frac{(Q_u)_m}{(Q_u)_n} t_s \left[1 - \frac{(Q_u)_m (f'_c)_n}{(Q_u)_n (f'_c)_m} \frac{\xi_3}{2}\right] + \frac{(F_y)_m}{(F_y)_n}\frac{d}{2} - \frac{(F_y)_m}{(F_y)_n}\xi_6 t_f - [\psi_1][\psi_2]}{\xi_4 t_s \left[1 - \frac{\xi_3}{2}\right] + \frac{d}{2} - \xi_6 t_f - [\psi_3][\psi_4]}\right\} \quad \text{(Eq. I.32)}$$

where:

$$\psi_1 = \frac{(F_y)_m}{(F_y)_n} - \frac{(Q_u)_m}{(Q_u)_n}\xi_4 - 2\frac{(F_y)_m}{(F_y)_n}\xi_6$$

$$\psi_2 = t_{tf} - \frac{b_f t_f}{2t_w} + \frac{A_s\left(1 - \xi_4 \frac{(Q_u)_m (F_y)_n}{(Q_u)_n (F_y)_m}\right)}{4t_w}$$

$$\psi_3 = 1 - \xi_4 - 2\xi_6$$

$$\psi_4 = t_{tf} - \frac{b_f t_f}{2t_w} + \frac{A_s(1 - \xi_4)}{4t_w}$$

$$V_M = \frac{\sqrt{\begin{array}{c}\left[\xi_4 t_s - \xi_3\xi_4 \frac{(Q_u)_m (f_c^{'})_n}{(Q_u)_n (f_c^{'})_m} - \xi_4\left(\frac{A_s}{2t_w} + t_f - \frac{b_f t_f}{t_w}\right) + \xi_4^2 \frac{A_s}{2t_w}\frac{(Q_u)_m (F_y)_n}{(Q_u)_n (F_y)_m}\right]^2 \sigma_{Q_u}^2 + \\ \left[\left(\frac{(Q_u)_m (f_c^{'})_n}{(Q_u)_n (f_c^{'})_m}\right)^2 \frac{t_s \xi_4}{2}\right]^2 \sigma_{f_c^{'}}^2 + \left[\frac{d}{2} - 3t_f\xi_6 + t_f - \frac{b_f t_f}{t_w} + \frac{A_s}{4t_w} + \frac{A_s\xi_6^2}{t_w} - \frac{\xi_4^2 A_s}{4t_w}\left(\frac{(Q_u)_m (F_y)_n}{(Q_u)_n (F_y)_m}\right)^2\right]^2 \sigma_{F_y}^2\end{array}}}{\xi_4 \frac{(Q_u)_m}{(Q_u)_n} t_s\left[1 - \frac{(Q_u)_m (f_c^{'})_n}{(Q_u)_n (f_c^{'})_m}\frac{\xi_3}{2}\right] + \frac{(F_y)_m}{(F_y)_n}\frac{d}{2} - \frac{(F_y)_m}{(F_y)_n}\xi_6 t_f - [\psi_1][\psi_2]} \quad \text{(Eq. I.33)}$$

Table I.11 shows a summary of all computed average resistance factors. The factors were computed for various conditions and sets of data to observe various trends. As can be seen, when the shear connectors governs, resistance factors are generally lower. Also, the difference between resistance factors corresponding to shear connectors and slab governing is more pronounced in tests with ribbed slabs. As the flexural theory is applied in the same manner to the beams with ribbed and the solid slabs, this difference is attributed to relatively high inaccuracy of Eqs. I.2 and I.3 (Table I.4).

Table I.10 ϕ for Solid Slab Beams, P.N.A. in Web, and Shear Connection Governing

SHAPE*	ξ_3	ξ_4	ξ_6	t_s	V_P	V_F	$(M_u)_m/(M_u)_n$	V_M	V_R	Φ	$\Phi_{average}$
W1100x449 (W44X335)	**0.212**	**0.212**	**0.288**	**160**	0.06	0.05	1.13	0.13	0.15	0.88	**0.89**
W1000x321 (W40X215)	**0.196**	**0.196**	**0.304**	**152**	0.06	0.05	1.13	0.13	0.15	0.88	
W920x223 (W36X150)	**0.245**	**0.245**	**0.304**	**152**	0.06	0.05	1.13	0.14	0.16	0.86	
W690x240 (W27X161)	**0.182**	**0.182**	**0.318**	**152**	0.06	0.05	1.13	0.12	0.15	0.88	
W530x82 (W21X55)	**0.235**	**0.235**	**0.265**	**152**	0.06	0.05	1.13	0.12	0.15	0.89	
W460x213 (W18X143)	**0.149**	**0.149**	**0.351**	**152**	0.06	0.05	1.13	0.12	0.15	0.89	
W360x1202 (W14X808)	**0.098**	**0.098**	**0.402**	**152**	0.06	0.05	1.13	0.13	0.15	0.88	
W360x1086 (W14X730)	**0.091**	**0.091**	**0.409**	**152**	0.06	0.05	1.13	0.13	0.15	0.88	
W250x149 (W10X100)	**0.108**	**0.108**	**0.392**	**152**	0.06	0.05	1.13	0.11	0.14	0.90	
M310x14.9 (M12X10)	**0.302**	**0.302**	**0.198**	**152**	0.06	0.05	1.14	0.12	0.14	0.90	
M150x5.5 (M6X3.7)	**0.263**	**0.263**	**0.237**	**150**	0.06	0.05	1.15	0.11	0.14	0.91	
S310x74 (S12X50)	**0.253**	**0.253**	**0.247**	**152**	0.06	0.05	1.14	0.12	0.14	0.90	
S75x11.2 (S3X7.5)	**0.203**	**0.203**	**0.297**	**76**	0.06	0.05	1.14	0.11	0.14	0.91	
S75x8.5 (S3X5.7)	**0.135**	**0.135**	**0.365**	**76**	0.06	0.05	1.14	0.11	0.14	0.91	

*Parenthesis contain the shape designations in English units.

Table I.11Summary of Computed Average ϕ Values

DESCRIPTION	ϕ SLAB/BEAM	ϕ SHEAR CONNECTION	ϕ AVERAGE
Solid Slabs – P.N.A. in web	0.87	0.87	0.87
Solid Slabs – P.N.A. in flange	0.89	0.89	0.89
Solid Slabs – P.N.A. in concrete slab	0.90	0.88	0.89
Solid Slabs - Average	**0.89**	**0.88**	**0.89**
Ribbed Slabs – P.N.A. in web (1999 AISCS)	0.81	0.79	0.80
Ribbed Slabs – P.N.A. in flange (1999 AISCS)	0.86	0.80	0.83
Ribbed Slabs – P.N.A. in concrete slab (1999 AISCS)	0.81	0.81	0.81
Ribbed Slabs (1999 AISCS) - Average	**0.83**	**0.80**	**0.81**
Ribbed Slabs – P.N.A. in web (R)	0.83	0.84	0.83
Ribbed Slabs – P.N.A. in flange (R)	0.91	0.93	0.92
Ribbed Slabs – P.N.A. in concrete slab (R)	0.92	0.89	0.90
Ribbed Slabs (R) - Average	**0.89**	**0.88**	**0.89**
Ribbed Slabs – P.N.A. in web (RS)	0.83	0.80	0.81
Ribbed Slabs – P.N.A. in flange (RS)	0.86	0.82	0.84
Ribbed Slabs – P.N.A. in concrete slab (RS)	0.89	0.89	0.89
Ribbed Slabs (RS) - Average	**0.86**	**0.83**	**0.85**
Both Types of Slabs – P.N.A. in web (1999 AISCS)	0.83	0.79	0.81
Both Types of Slabs – P.N.A. in flange (1999 AISCS)	0.87	0.82	0.84
Both Types of Slabs – P.N.A. in concrete slab (1999 AISCS)	0.87	0.87	0.87
Both Types of Slabs (1999 AISCS) - Average	**0.86**	**0.83**	**0.84**
Both Types of Slabs – P.N.A. in web (R)	0.85	0.86	0.85
Both Types of Slabs – P.N.A. in flange (R)	0.92	0.93	0.92
Both Types of Slabs – P.N.A. in concrete slab (R)	0.91	0.89	0.90
Both Types of Slabs (R) - Average	**0.89**	**0.89**	**0.89**
Both Types of Slabs – P.N.A. in web (RS)	0.84	0.81	0.82
Both Types of Slabs – P.N.A. in flange (RS)	0.87	0.83	0.85
Both Types of Slabs – P.N.A. in concrete slab (RS)	0.90	0.89	0.90
Both Types of Slabs (RS) - Average	**0.87**	**0.84**	**0.86**

As described earlier, and as illustrated by Fig. I.1, it was expected that the pool of test data associated with lower degree of shear connection, which in turn is generally associated with the beams with P.N.A. in the web, would reflect lower values of resistance the factor.

This was confirmed in the analysis, and it can be seen from information in Table I.11 that the tests with P.N.A. in the web (i.e. with lower degree of shear connection) yield a comparatively lower resistance factor.

If the 1999 AISCS method is considered, and if absolutely no distinction is made between tests with ribbed slabs and those with solid slabs, and if the location of P.N.A. and the degree of shear connection is disregarded for the purpose of reliability analysis, an average ϕ of 0.86 is computed for beams considering that either steel beam, or concrete slab, as applicable, govern. This drops to 0.83 if shear connection governs, resulting in an average of 0.84 between the two. The average resistance factor, however, drops to 0.81 if the ribbed-slab tests only are considered. For comparison, the average of 0.85 is computed if RS-method is used and ribbed slabs are considered. As a direct result of improved accuracy of shear connection strength calculation model, the resistance factor of 0.89 is calculated for both when slab/beam and shear connection govern, if the R-method is used. Using an absolute average value, however, may not be completely justified. As shown in Table I.11, ϕ varies, depending on condition considered, from 0.79 to 0.90. Also, these values were observed through the analysis to get as high as 0.96 for specific extreme cases, and as low as 0.76. Therefore, it would be prudent to address the issue so that similar reliability at all degrees of shear connection and for both types of slab is achieved.

The above given values of combined resistance factors were obtained by calculating separate resistance factors for each location and P.N.A. The computed values were then combined into a relative average. In contrast to that approach, it is possible to compute combined means and C.O.V.s of the entire test population, regardless of the location of the P.N.A., type of slab, or both. The overall resistance factor can then be reported based on average influence of Eqs. I.13 through I.33. If that is done in this case, ϕ of 0.87 is computed for solid slabs, while ϕ values of 0.82, 0.87 and 0.83 were computed for ribbed-slab tests corresponding to 1999 AISC, R, and RS methods, respectively. If the statistical parameters of solid and ribbed slabs are combined, the ϕ values of 0.84, 0.87, and 0.85 were computed for 1999 AISC, R, and RS method, respectively. However, this method of resistance factor computation is not recommended, because it was found to be unrealistically sensitive and biased with respect to the sample size.

As most of variability is associated with the 1999 AISC method, it will not be considered further and the focus will be on R and RS-methods. The first possible solution is to establish a simple relationship that can be used to vary the resistance factor as a function of the degree of shear connection. The relationship between the resistance factor and the degree of shear connection in solid slabs is shown in Fig. I.4. Obviously, there is very little difference in the resistance factor with respect to the degree of shear connection. This is a direct consequence of relatively conservative nature of shear strength prediction model for solid slabs, as reflected in Table I.4, so a uniform resistance factor of 0.90 can be safely used for solid slabs at all degrees of shear connection. The relationship between resistance factor and degree of shear connection for ribbed-slab tests when R and RS-methods, respectively, are used is illustrated in Figs. I.5 and I.6.

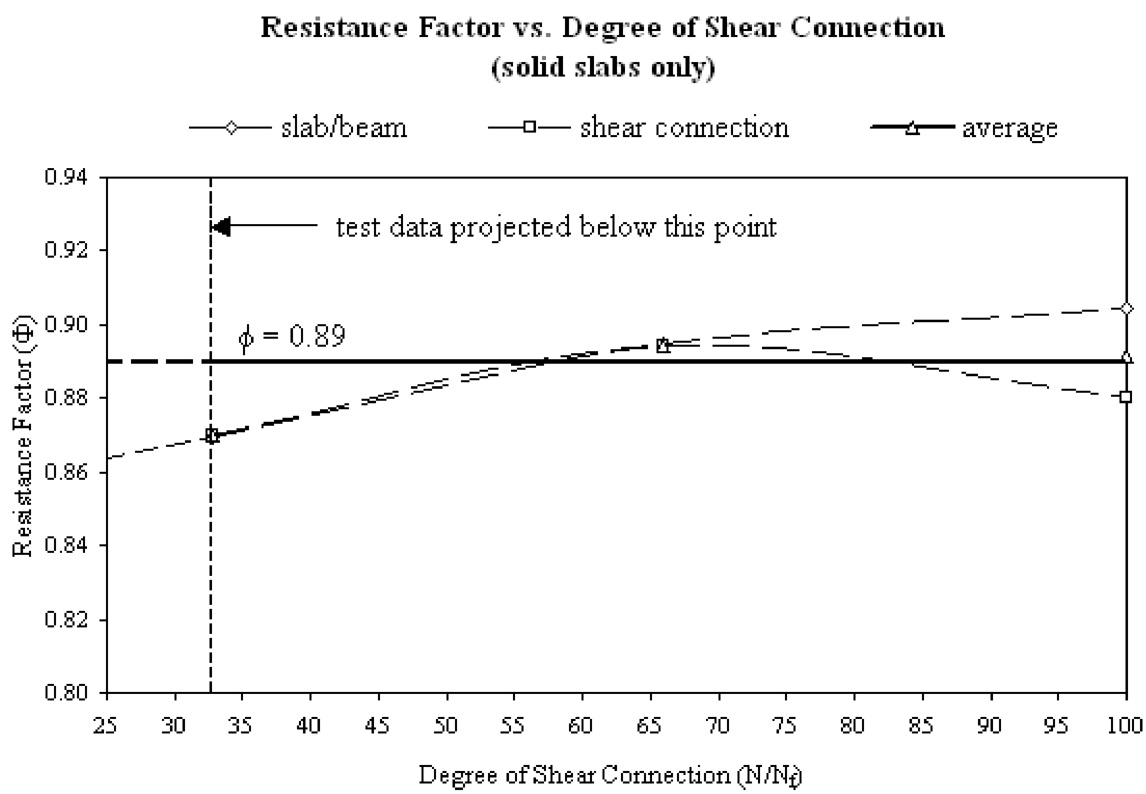

Fig. I.4 Variation of ϕ with the Degree of Shear Connection (Solid Slabs)

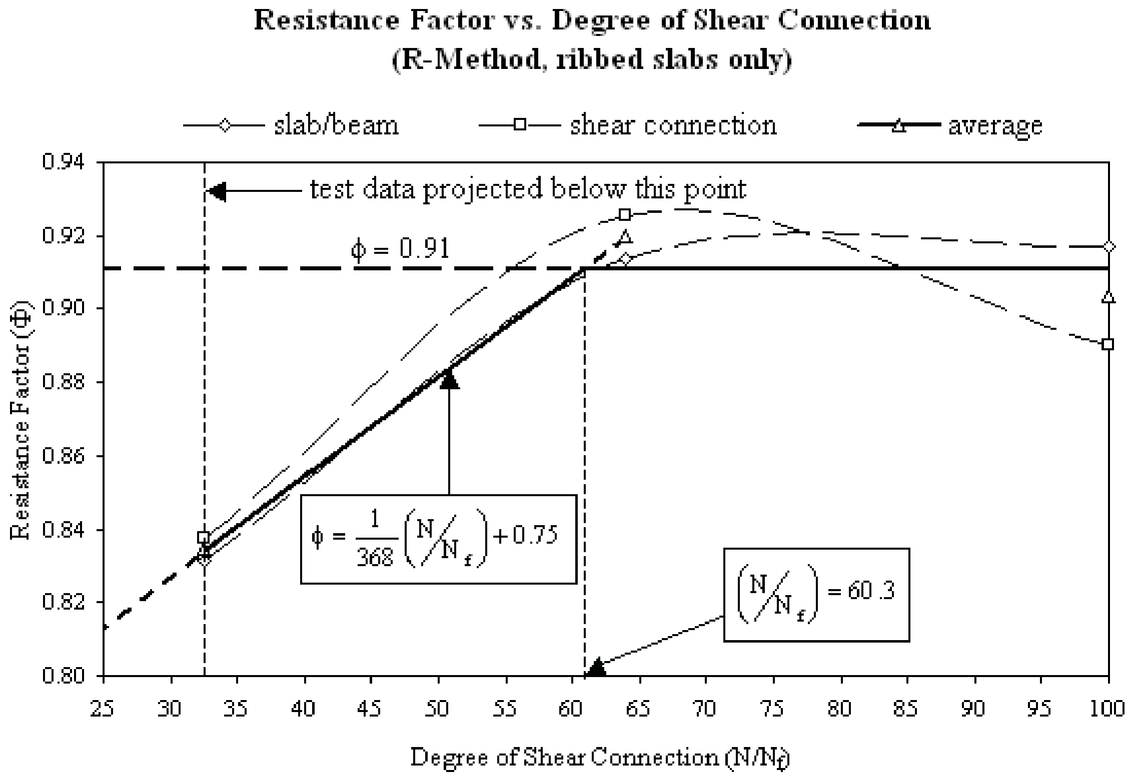

Fig. I.5 Variation of ϕ with the Degree of Shear Connection (R-Method)

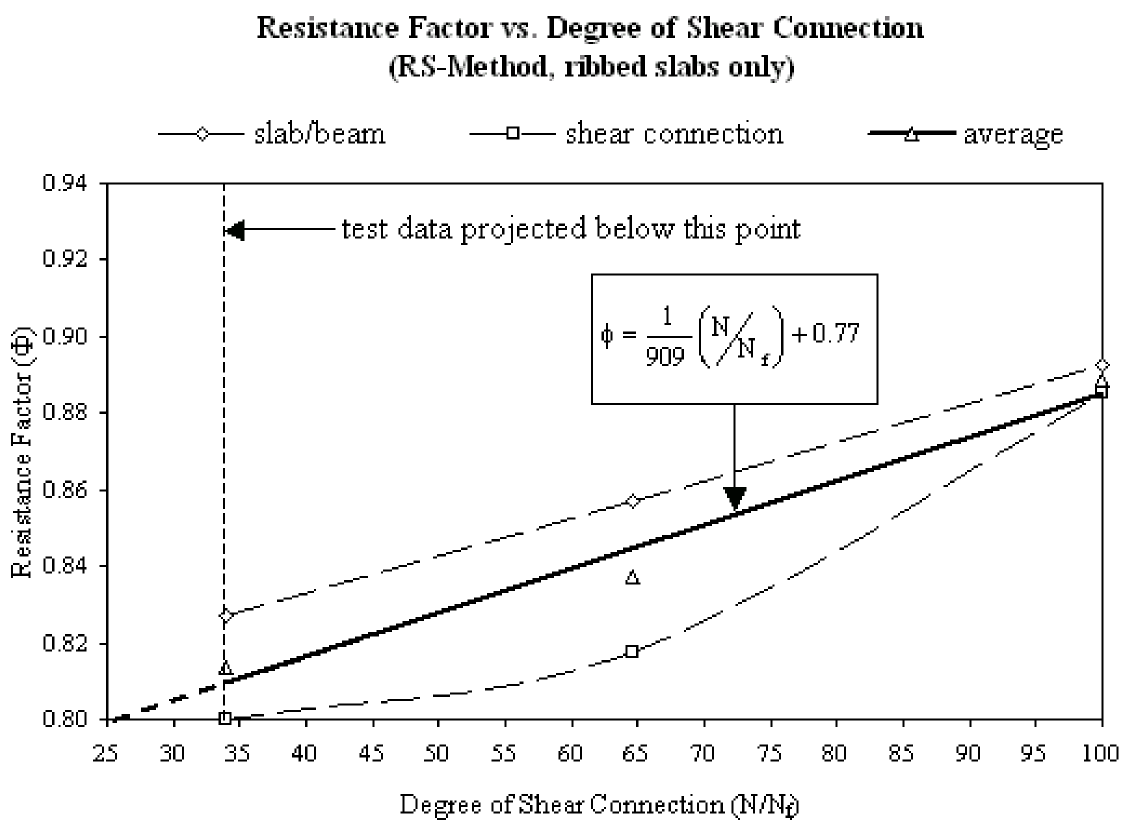

Fig. I.6 Variation of ϕ with the Degree of Shear Connection (RS-Method)

With respect to the R-method, it can be seen that the relationship between the resistance factor and the degree of shear connection varies significantly between 25 and about 60 percent of composite action, and is then nearly constant above 60 percent. If the relationship shown in Fig. I.5 is further expanded to also include solid slab beams, the overall resistance factor for composite beams can be expressed as follows:

$$\phi = 0.90\nu \qquad \text{(Eq. I.35)}$$

where:

$\nu = 1.0$ for solid slab beams and ribbed slab beams with degree of shear connection of at least 55 %

$\nu = \frac{1}{370}\left(N/N_f\right) + 0.75$ for beams ribbed slabs of less than 55 % composite action

$\left(N/N_f\right)$ = degree of shear connection, $\{25 \leq N/N_f \leq 100\}$

With respect to the RS-method, it can be see that the relationship between the resistance factor and the degree of shear connection is nearly linear. The relationship shown can be further expanded to include solid slabs. Equation I.36 shows the unified expression for the resistance factors to be applied to the beams with solid slabs and ribbed slabs with ribs perpendicular to the member.

$$\phi = 0.90\nu \qquad \text{(Eq. I.36)}$$

where:

$\nu = 1.0$ for solid slab beams

$\nu = \frac{1}{889}\left(N/N_f\right) + 0.79$ for beams ribbed slabs

$\left(N/N_f\right)$ = degree of shear connection, $\{25 \leq N/N_f \leq 100\}$

As previously noted, the focus of this paper was on beams with solid slabs, and slabs with ribs perpendicular to the member. Beams with slabs where ribs are parallel to the slab were not treated herein due to a lack of test data. Also, these types of composite beams are generally not subject to concerns prevalent for slabs with perpendicular ribs. Pending further experimental verifications, the resistance factor of 0.90 for solid slabs can be safely applied to slabs with parallel ribs. The subsequent verifications of such beams should focus on cases where $w_r/h_r < 1.5$. The types of beams are behaviorally identical with respect to development of flexural strength, and solid slab tests were used in the past to model the strength of shear connection for similar shear connectors in slabs with parallel ribs (Mujagic et al. 2001).

Finally, the second possible way to address the issue of variability of shear connection with degree of shear connection is to raise the minimum required percentage of shear connection to the level at which resistance factor does not appreciably differ from that at fully composite beams. A rational choice would be to raise the minimum required degree of shear connection to 50 %. There are also other reasons for which this would be advisable, such as control of slip, deflection and member and shear connection ductility. Specifically, Eurocode 4 (CEN 1992) stipulates a minimum percentage of shear connection as a function of the member length. In addition to member length, the required minimum percentage depends on various geometric parameters, and is never less than 40%. Previous studies on solid slab beams (McGarraugh and Baldwin 1971) have concluded that the minimum required degree of shear connection to adequately control slip and deflections should in no instance be less than 50%.

I.4 SUMMARY AND CONCLUSIONS

The presented study evaluates the reliability of composite beams and proposes revised resistance factors based on proposed revisions to the provisions of current 1999 AISC Specification. The resistance factors were also found based on newly proposed models to calculate the strength of shear connection: Roddenberry et al. model, and Simplified Roddenberry model. It was found that the resistance factor for solid slab beams of 0.90 could be used. If 1999 AISC provisions are used, and ribbed-slab beams are considered, the resistance factor ranges from 0.79 to 0.86, with an average of about 0.80, showing that combining the statistics from the two types of tests and a uniform resistance factor is not justified with this method. The situation is appreciably improved with RS-method, by which the currently stipulated resistance factor of 0.85 for ribbed slabs is actually achieved. If the

R-method was used, a uniform resistance factor of 0.90 may be appropriate for all types of slabs.

Finally, it was shown that degree of shear connection has significant impact on the resistance factor. Functions were proposed that could be used to calculate the resistance factor based on degree of shear connection in a beam. Alternatively, it is proposed that the minimum degree of shear connection be raised to 50 % and a uniform resistance factor of 0.90 be used provided that either R or RS method is adopted.

I.5 REFERENCES

ACI (2002). *Building Code Requirements for Structural Concrete (ACI 318-02) and Commentary (ACI 318R-02)*. American Concrete Institute, Farmington Hills, Michigan.

AISC (1999). *Load and Resistance Factor Design Specifications for Structural Steel Buildings*. American Institute of Steel Construction, Chicago, Illinois.

AISC (2001). *Manual of Steel Construction: Load and Resistance Factor Design*, 3rd Ed. American Institute of Steel Construction, Chicago, Illinois.

Allan, B., Yen, B. T., Slutter, R. G., and Fisher, J. W. (1976). "Comparative Tests on Composite Beams with Formed Metal Deck." Fritz Engineering Laboratory Report No. 200.76.458.1. Lehigh University, Bethlehem, PA.

Baldwin, J.W., Henry, J.R., and Sweeney, G.M. (1965). "Study of Composite Bridge Stringers: Phase II." Research Report, University of Missouri, Columbia, MO.

Barnard, P.R., and Johnson, R.P. (1965). "Ultimate Strength of Composite Beams." Paper No. 6836, Institution of Civil Engineers Proceedings, 161-179.

Bartlett, F.M., Dexter, R.J., Greaser, M.D., Jelinek, J.J., Schmidt, B.J., and Galambos, T.V. (2003). "Updating Standard Shape Material Properties Database for Design and Reliability." Engineering Journal, AISC, 1st Quarter, Vol. 40, No. 1, 2-14.

Bethlehem Steel Corporation. (1965). "Slabform in Composite Construction." Bethlehem Steel Corporation, Bethlehem, PA.

CEN (1992). "ENV 1994-1-1: Design of composite steel and concrete structures – Part 1.1: General rules and rules for buildings." Brussels, Belgium.

Cornell, C.A. (1969). "A Probability-Based Structural Code." *Journal of the American Concrete Institute*, ACI, 66(12), 974-985.

Culver, C., and Coston, R. (1961). "Tests of Composite Beams with Stud Shear Connectors." *Journal of the Structural Division*, ASCE, Vol. 87, No. ST2, 1-17.

Diaz, B., Easterling, W.S. and Murray, T.M. (1998). "Strength of Welded Headed Studs Used with 1.0C Deck." Internal Report, Virginia Polytechnic Institute and State University, Blacksburg, VA.

Easterling, W.S., Gibbins, D.R. and Murray, T.M. (1993). "Strength of Shear Studs in Steel Deck on Composite Beams and Joists," *Engineering Journal*, AISC, Vol. 30, No. 2, 2nd Quarter, 44-55.

Errera, S. J. (1967). "Analysis of Tests of Composite Steel and Concrete Beams with Mahon Steel Decking." Structural Engineering Department, Cornell University, Ithaca, NY. December.

Fisher, J. W., Kim, S. W., and Slutter, R. G. (1967). "Tests of Lightweight Concrete Composite Beams and Pushout Specimens with Cellular Steel Deck." Fritz Engineering Laboratory Report No. 200.67.438.1. Lehigh University, Bethlehem, PA.

Fisher, J.W. (1973). "Test of Composite Beam with Lock-Form Flutted and Cellular Units and Lightweight Concrete Slab." Fritz Engineering Laboratory Report. Lehigh University, Bethlehem, PA.

Furlong, R. W. and Henderson, W. D. (1975). "Report of Load Tests on Composite Beams of Lightweight Concrete in Three-Inch Metal Deck with Stud Length as the Principle Variable." University of Texas at Austin.

Galambos, T.V., and Ravindra, M.K. (1973). Tentative Load and Resistance Factor Design Criteria for Steel Buildings. Research Report No. 18, Washington University, St. Louis, MO.

Galambos, T.V. and Ravindra, M.K. (1976). Load and Resistance Factor Design Criteria for Composite Beams. Research Report No. 44, Washington University, St. Louis, MO.

Galambos, T.V. and Ravindra, M.K. (1978). "Properties of Steel for Use in LRFD." *Journal of the Structural Division*, ASCE, 104(9), 1459-1468.

Grant, J.A., Fisher, J.W. and Slutter, R.G. (1977). "Composite Beams with Formed Steel Deck." *Engineering Journal*, AISC, Vol. 14, No.1, 1st Quarter, 24-43.

Hansell, W.C., Galambos, T.V., Ravindra, M.K. and Viest, I.M. (1978). "Composite Beam Criteria in LRFD." *Journal of the Structural Division*, ASCE, 104(9), 1409-1426.

Hanson, R.E. (1970). "Full Scale Beam Test of Stud Shear Connectorsand C2 Cofar." Report No. T-1511A, Granco Steel Products Co., St. Louis, MO.

Inland Steel Product Company. (1967). "Certification Tests for the First National Bank of Chicago." Report No. 1, Inland Steel Product Company, Milwaukee, WI.

Jayas, B. S., and Hosain, M. U. (1989). "Behavior of Headed Studs in Composite Beams: Full-Size Tests." *Can. J. Civ. Eng.*, 16(5), 712-724.

Johnson, R.P. and Anderson, D. (1993). *Designers' Handbook to Eurocode –Part 1.1: Design of composite steel and concrete structures*, First Edition, Thomas Telford, London, UK.

Johnson, R.P. (1994). *Composite Structures of Steel & Concrete*, Vol. 1, Second Edition. Blackwell Scientific Publications, Oxford, UK.

Jones, B. T. (1966). "The Flexural Strength of Steel and Concrete Composite Beams Using Cellular Floor and Q-Lock Floor with Stud Shear Connectors." Report No. 66-4,5, H. H. Robertson Co., Pittsburgh, PA. September.

Jones, B. T. (1975). "Composite Beam Test '3 + 2' BPTC Technical Data Report No. 75-16." H. H. Robertson Co., Pittsburgh, PA.

Lacap, D. A. (1975). "Composite Beam Tests (W24x61 and W24x55) for Columbia University Life Science Bldg., New York, NY. Test Nos. 174-75 and 175-75." Inland-Ryerson Construction Products Co. Milwaukee, WI.

Lacap, D. A. (1976). "Composite Beam Test W21x44 – 3V – 30 in. and 3NF – 30 in. 20 Gage Decks, 5.5 in. Regular Weight Concrete Slab for Metropolitan Life, Warwick, RI. Test No. 16-76. Inland-Ryerson Construction Products Co. Milwaukee, WI.

Lew, H.S. (1970). "Effect of Shear Connector Spacing on the Ultimate Strength of Concrete-on-Steel Composite Beams." Report 10-246. National Bureau of Standards.

Lyons, J.C., Easterling, W.S. and Murray, T.M. (1994). "Strength of Welded Shear Studs." Report No. CE/VPI - ST 94/07, Virginia Polytechnic Institute and State University, Blacksburg, VA.

McGarraugh, J. B. and Baldwin, J.W. (1971). "Lightweight Concrete-on-Steel Composite Beams," *Engineering Journal*, AISC, Vol. 8, No. 3, 3rd Quarter, 90-98.

Michalski, K.J., Thelen, A.B., Weiler, T.J. (1970). "Final Report of Typical Composite Beam Test for McGraw-Hill Office Building," Job No. 71864, Research and Development Laboratory of the Inland-Ryerson Products Company, Milwaukee, WI.

Mujagic, J.R.U., Easterling, W.S. and Murray, T.M. (2001). "Strength Calculation Model for Standoff Screws in Composite Joists." Report No. CE/VPI - ST 00/19, Virginia Polytechnic Institute and State University, Blacksburg, VA.

Mujagic, J.R.U. and Easterling, W.S. (2003). "Reliability Assessment of Composite Beams: Data Report." Report No. CE/VPI - ST 03/11, Virginia Polytechnic Institute and State University, Blacksburg, VA.

Nelson Stud Welding Company. (1971a). "Tensile Tests: Nelson Shear Connector Studs Welded Through Hot-Dip Galvanized Steel Sheet." Test Report 1971-5, Herron Testing Laboratories, Inc., Cleveland, OH.

Nelson Stud Welding Company. (1971b). "Tensile Tests: Nelson Shear Connector Studs Welded Through Wiped Coat Galvanized Sheet Steel." Test Report 1971-6, Herron Testing Laboratories, Inc., Cleveland, OH.

Nelson Stud Welding Company. (1972). "AWS Qualification Tests for ¾" Diameter Stud Base and #100-101-175 Arc Shield." Test Report 1972-14, Herron Testing Laboratories, Inc., Cleveland, OH.

Poletto, R.J., Corrado, J.A., Slutter, R.G. (1970). "Flexure and Push-out Tests of Composite Steel –Lightweight Concrete Specimens with Rollform Type B Metal Decking." Report No. 200.69.81.1, Lehigh University, Bethlehem, PA.

Proctor, M.H. (1963). "Analysitical and Experimental Study of Lightweight Concrete-Steel Composite Beams." M.S. Thesis, University of Missouri, Columbia, MO.

Rambo-Roddenberry, M., Easterling, W.S. and Murray, T.M. (2002a). "Behavior and Strength of Welded Stud Shear Connectors." Report No. CE/VPI - ST 02/04, Virginia Polytechnic Institute and State University, Blacksburg, VA.

Rambo-Roddenberry, M., Easterling, W.S. and Murray, T.M. (2002b). "Behavior and Strength of Welded Stud Shear Connectors – Data Report." Report No. CE/VPI - ST 02/05, Virginia Polytechnic Institute and State University, Blacksburg, VA.

Ravindra, M.K., Heaney, A.C., and Lind, N.C. (1969). "Probabilistic Evaluation of Safety Factors," Symposium on Concepts of Safety of Structures and Methods of Design, IABSE, London, UK.

Rivera, U.C., and Slutter, R.G. (1968). "Tests of Lightweight Concrete Composite Beams with Metal Decking." Report No. 200.67.458.1, Lehigh University, Bethlehem, PA.

Robinson, H. (1969). "Composite Beam Incorporating Cellular Steel Decking." *Journal of the Structural Division*, ASCE, Vol. 95, No. ST3.

Robinson, H. and Wallace, I. W. (1971). "Composite Beams with Partial Connection." Meeting preprint No. 1549, ASCE Annual and National Environmental Engineering, St. Louis, MO. October 18-22.

Robinson, H. (1988). "Multiple Stud Shear Connectors in Deep Ribbed Metal Deck." *Can. J. Civ. Eng.*, 15(4), 553-569.

Seek, W. G., Fisher, J. W., and Slutter, R. G. (1970). "Tests of Lightweight Composite Beams with Metal Decking." Fritz Engineering Laboratory Report No. 200.70.458.1. Lehigh University, Bethlehem, PA.

Skiner, T.A. (1971). "Shear Connector Behavior from Beam Tests: Phase II." M.S. Thesis, University of Missouri, Columbia, MO.

Slutter, R.G., Driscoll, G.C. (1965). "Flexural Strength of Steel-Concrete Composite Beams." *Journal of the Structural Division*, ASCE, Vol. 91, No. ST2.

Slutter, R.G. (1969a). “Tests of Lightweight Concrete Members with Metal Decking.” Report No. 200.68.458.1, Lehigh University, Bethlehem, PA.

Slutter, R.G. (1969b). “Tests of Lightweight Concrete Members with Metal Decking.” Report No. 200.69.458.2, Lehigh University, Bethlehem, PA.

Slutter, R.G., and Adams, R.G. (1964). "Tests of Composite Beams with Holorib Composite Slabs." Report No. 200.63.408.2, Lehigh University, Bethlehem, PA.

Slutter, R.G. (1968). “Test of Cincinnati Center Composite Beam.” Report No. 200.67.458.2, Lehigh University, Bethlehem, PA.

Slutter, R.G. (1969a). “Tests of Lightweight Concrete Members with Metal Decking.” Report No. 200.68.458.1, Lehigh University, Bethlehem, PA.

Slutter, R.G. (1969b). “Tests of Lightweight Concrete Members with Metal Decking.” Report No. 200.69.458.2, Lehigh University, Bethlehem, PA.

Slutter, R.G., and Adams, R.G. (1971). "Tests of Composite Beams with Epicore Floor Systems and Lightweight Concrete Slab." Report, Lehigh University, Bethlehem, PA.

Sublett. C.N., Easterling, W.S. and Murray, T.M. (1992). "Strength of Welded Headed Studs in Ribbed Metal Deck on Composite Joists." Report No. CE/VPI - ST 92/03, Virginia Polytechnic Institute and State University, Blacksburg, VA.

Thelen, A.B. (1972). “Composite Beam Test with 2 and 3, Φ3/4 in. x 4.5 in., Studs per Rib in 3V and Lightweight Concrete.” Report, Inland-Ryerson Construction Products Co., Milwaukee, WI.

Toprac, A. A. (1965). “Strength of Three New Types of Composite Beams,” *Engineering Journal*, AISC, Vol. 2, No. 1, 1st Quarter, 21-30.

Vogel, G. (1971). "Shear Connector Behavior from Beam Tests: Phase I." M.S. Thesis, University of Missouri, Columbia, MO.

Yam, L.C.P., and Chapmann, J.C. (1969). "The Inelastic Behaviour of Simply Supported Composite Beams of Steel and Concrete." *The Institution of Civil Engineers, Proceedings*, 651-683.

Yuan, H. (1996). "The Resistances of Shear Stud Connectors with Profiled Sheeting." Ph.D. Dissertation, University of Warwick, U.K.