Single-Plate Framing Connections with Grade-50 Steel and Composite Construction

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Single-plate framing connections, such as shown in Fig. 1, are widely used because of their economy of material and ease of construction. References 1, 2 and 3 present design guides that ensure their ductility and account for their moment resistance when used in symmetrical connections or with stiff supporting structures as in Fig. 1. These design procedures are applicable to connections with A307, A325 and A490 bolts used with A36 noncomposite steel beams. Research completed at the University of Arizona has expanded these design guides to include (a) connections with slotted holes, (b) connections with Gr. 50 steel beams, (c) off-axis framing plates (Fig. 2) and (d) single-plate framing connections with simply supported shored and unshored composite beams.

FULL SCALE NONCOMPOSITE BEAM TESTS

In Ref. 1, Richard reported the results of five full-scale tests of beams with single plate framing connections. The moment resisted by these connections was compared to the moment predicted by a simplified design curve, beam line analysis, and program INELAS, a nonlinear finite element analysis program.[7] This study continues those tests by investigating the following two common design practices: (1) use of slotted holes that aid in erection and (2) off-axis bolt groups (Fig. 2).

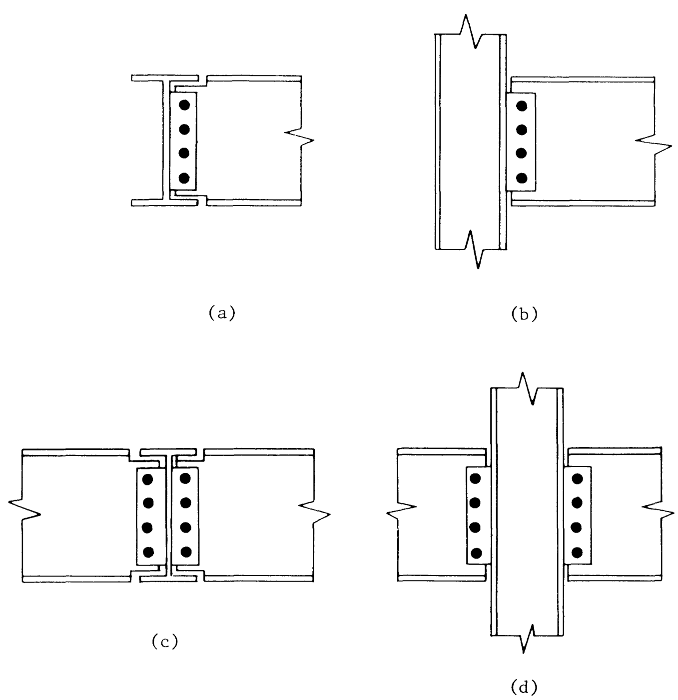

Fig. 1. Single-plate framing connections

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Beam Eccentricity

The parameter used to compute the connection moment is the beam eccentricity, defined as the horizontal distance from the centerline of the bolt group to the inflection point of the beam (see Fig. 3). Using this measure allows the distance from the bolt line to the weldment, a, to vary. Having measured or calculated e, the moment at the weldment is then

$$M = (e + a)R_{beam}$$

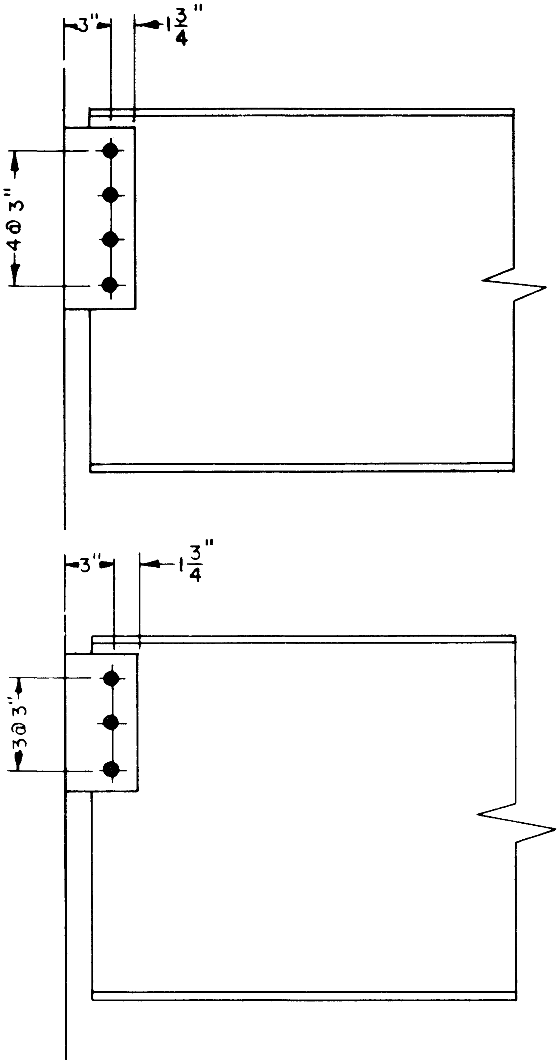


Fig. 2. Off-axis bolt groups

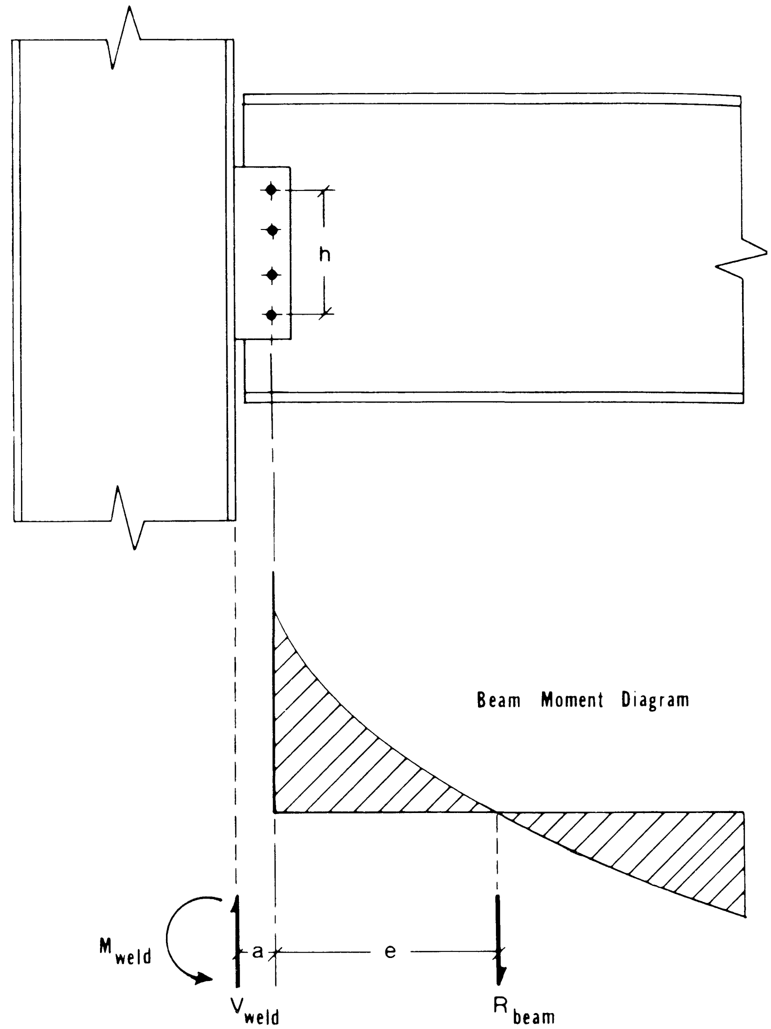


Fig 3. Definition of eccentricity

Test Procedure

Test numbers three through ten in Table 1 were run, using the apparatus shown in Fig. 4. Tests one and two were reported in Ref. 1 by Richard and are included here for comparison. The test beam was a 32-ft long W24x55 section connected at one end by a ⅜-in. A36 single framing plate to a rigid backup structure and simply supported at the other end. A325 grade bolts were tightened by the turn-of-the-nut method. A 50-ton jack was used to load the beam in increments up to 1.5 times the working load. The eccentricity was measured with strain gauges on the top and bottom beam flanges and rotations were measured with four dial gauges as shown in Fig. 4. The eccentricities at 1.5 times working load are summarized in Table 1.

Connections with Slotted Holes

The behavior of connections with slotted holes and A325 bolts torqued by the turn-of-the-nut method differ little from the behavior of the same connections with round holes due to the clamping action of the high-strength bolts.* Figures 5, 6, 7 and 8 show the eccentricities for round holes vs. slotted holes for off-axis and symmetrical connections at 1.5 times working load. In all the tests with A325 bolts in slotted holes (Tests 5–10), the calculated design curve eccentricities compared favorably with the measured eccentricities. When the connections were dismantled, there was no apparent distortion of the bolts or bolt holes and there was no perceptible bolt slip except in the three-bolt connection with 6-in. pitch where the dial gauges did indicate some slip.

*For design purposes, the reader is reminded that the required bolt diameter-to-plate thickness ratio for standard holes is waived for the case of slotted holes.

Table 1. Full-Scale Test Schedule and Eccentricities (in.)

Bolts	Test No.	No. of Bolts	Hole & Pattern	Beam Line Theory	Design Curve	Experimental Results
7/8 in. A325	1	7	Round concentric	48.0	49.5	42.2
	2	3	Round concentric (6 in. sp)	17.3	14.2	19.3
	3	3	Round off-axis	8.0	7.1	8.6
	4	4	Round off-axis	17.1	14.1	17.0
	5	7	Slotted concentric	48.0	49.5	33.1
	6	5	Slotted concentric	39.8	23.6	26.8
	7	3	Slotted concentric	8.0	7.1	9.8
	8	3	Slotted concentric (6 in. sp.)	17.3	14.2	12.0
	9	4	Slotted off-axis	17.1	14.1	15.0
	10	3	Slotted off-axis	8.0	7.1	9.0

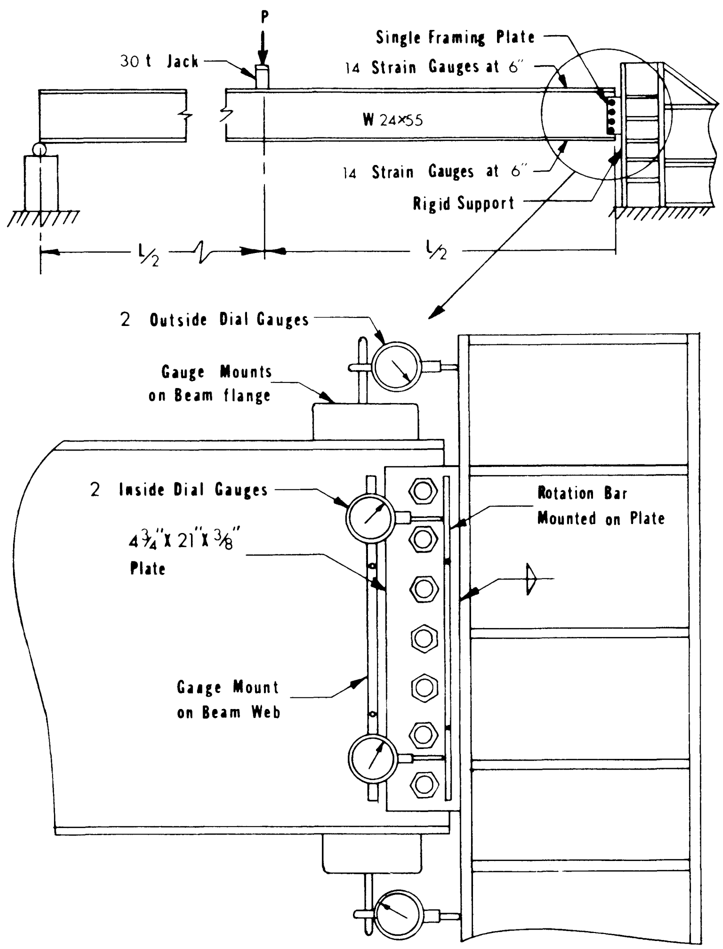


Fig. 4. Full-scale test apparatus

Off-axis Connections

Four A325 bolt connections were tested with off-axis bolt groups. As shown in Table 1, the eccentricities for off-axis bolt groups varied from those for symmetrical connections by, at most, ±9%. More importantly, the center of rotation for the off-axis connections was at the center of the bolt group as shown in Figs. 9 and 10. This was true for all off-axis connections tested.

Conclusions

Test results support the following conclusions:

1. Tightened A325 and A490 bolts behave essentially the same in round or slotted holes.
2. The moment-rotation response of a single plate framing connection is unaffected by the location of the neutral axis of the beam it supports.
3. The design curve presented in Ref. 1 satisfactorily predicts the eccentricity of single plate connections with off-axis tightened high-strength bolts. The same curve can be used to conservatively calculate eccentricity for connections with torqued high-strength bolts in slotted holes.

BEAMS OF $F_y = 50$ STEEL

The design curve reported in Ref. 1 for beams of $F_y =$ 36 steel was developed in part from beam line analysis. In this analysis the end conditions on the beam are assumed to vary linearly from zero rotation with fixed end moment to pinned end rotation with zero moment as

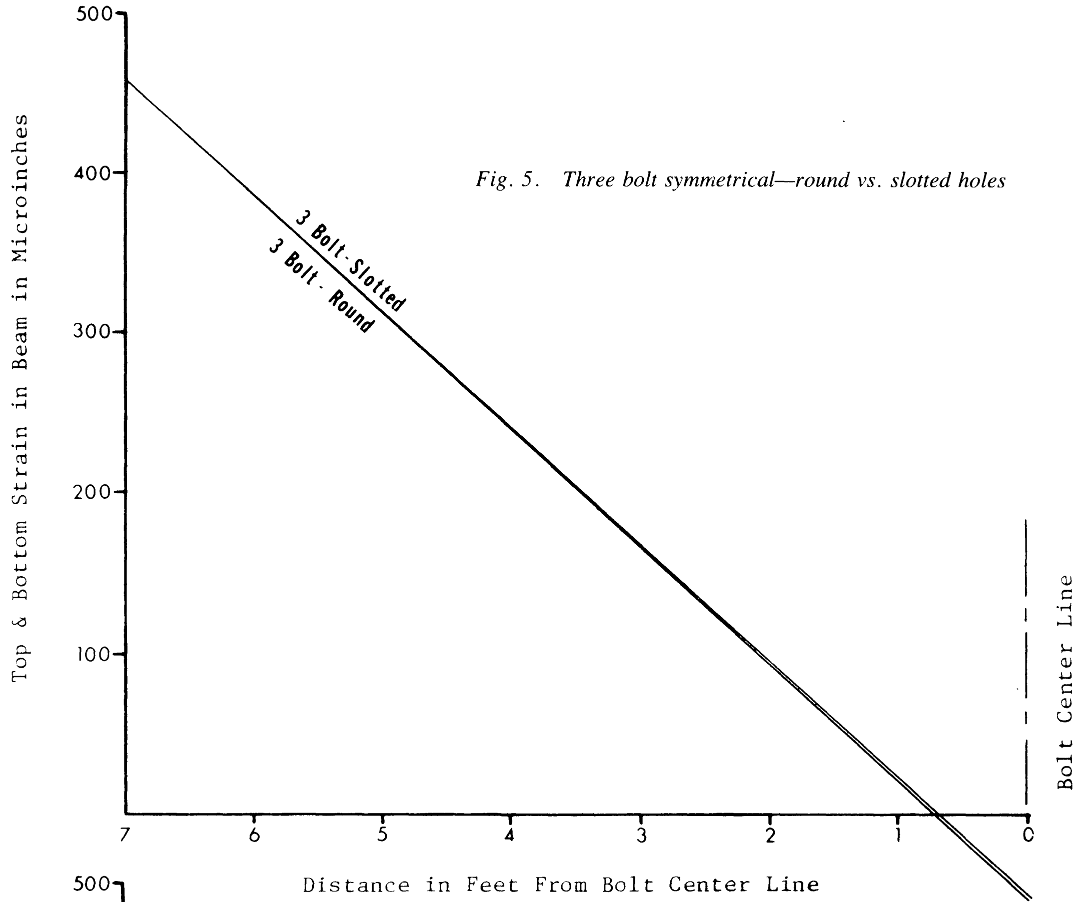


Fig. 5. Three bolt symmetrical—round vs. slotted holes

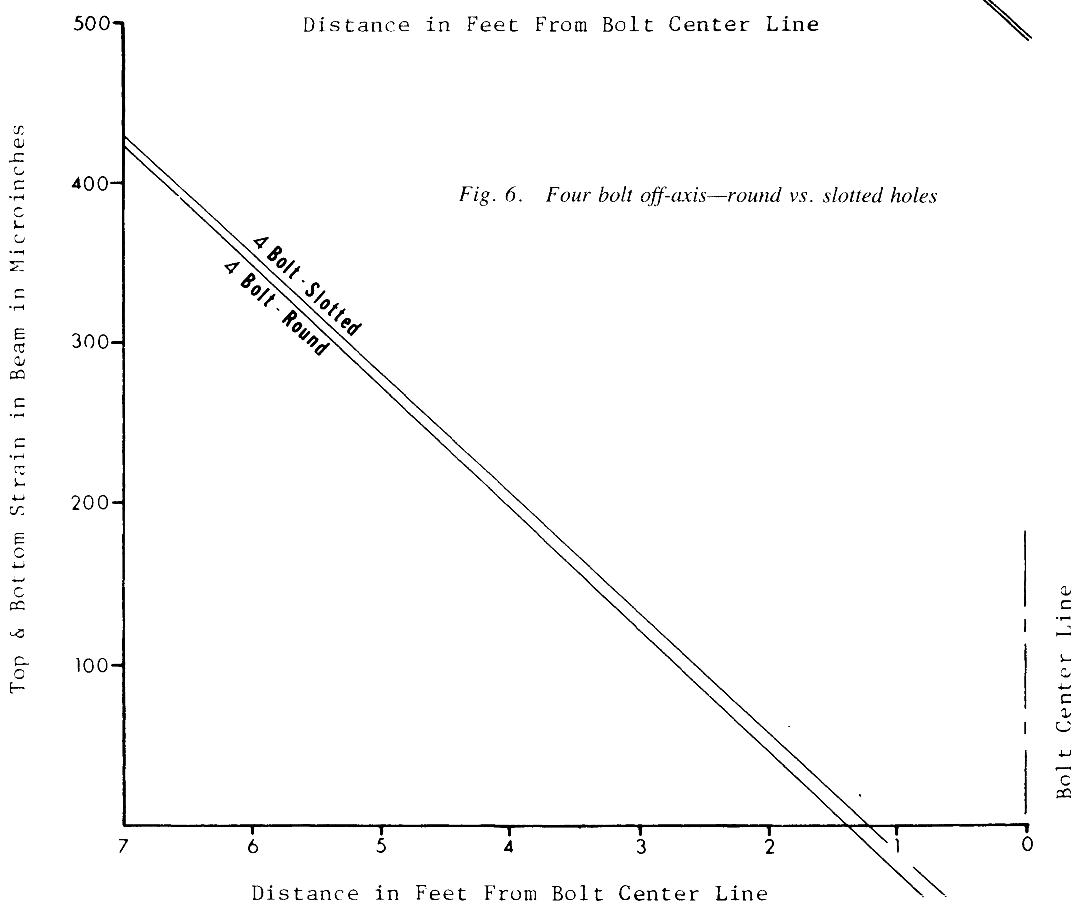


Fig. 6. Four bolt off-axis—round vs. slotted holes

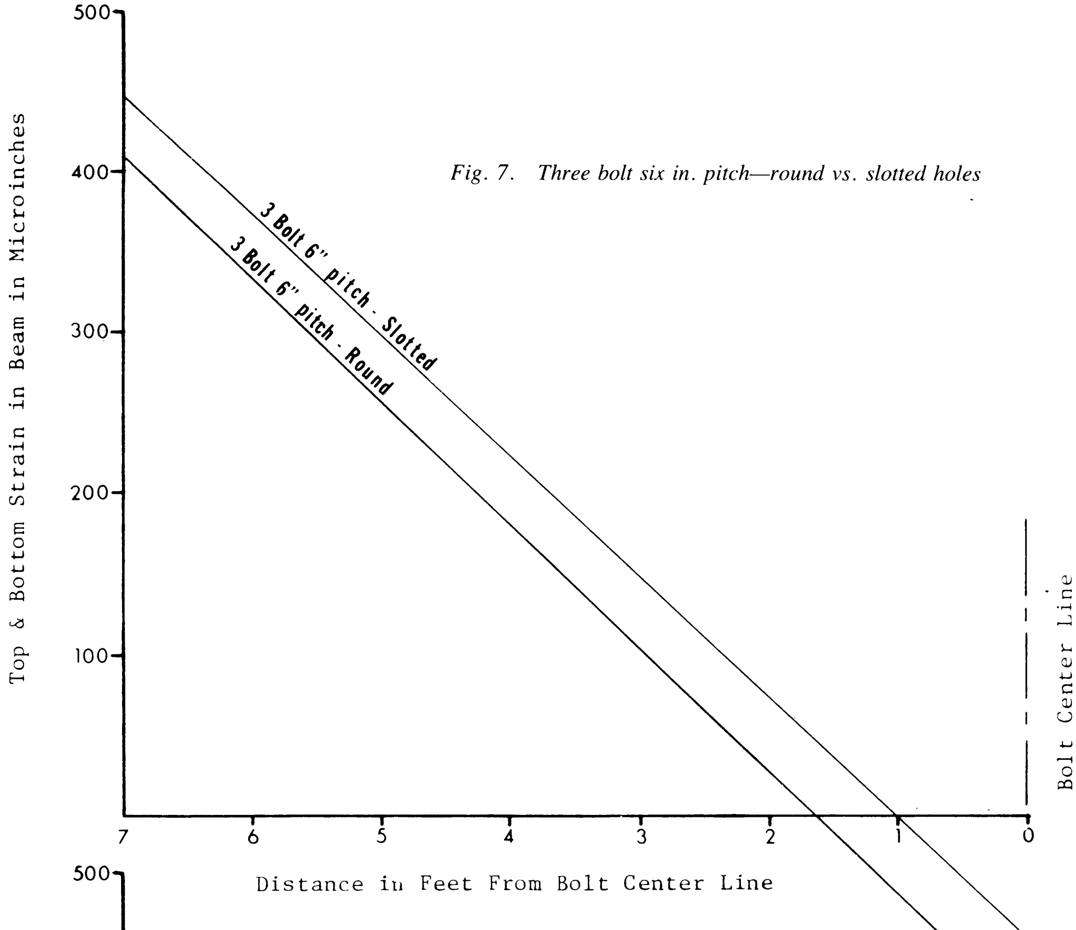


Fig. 7. Three bolt six in. pitch—round vs. slotted holes

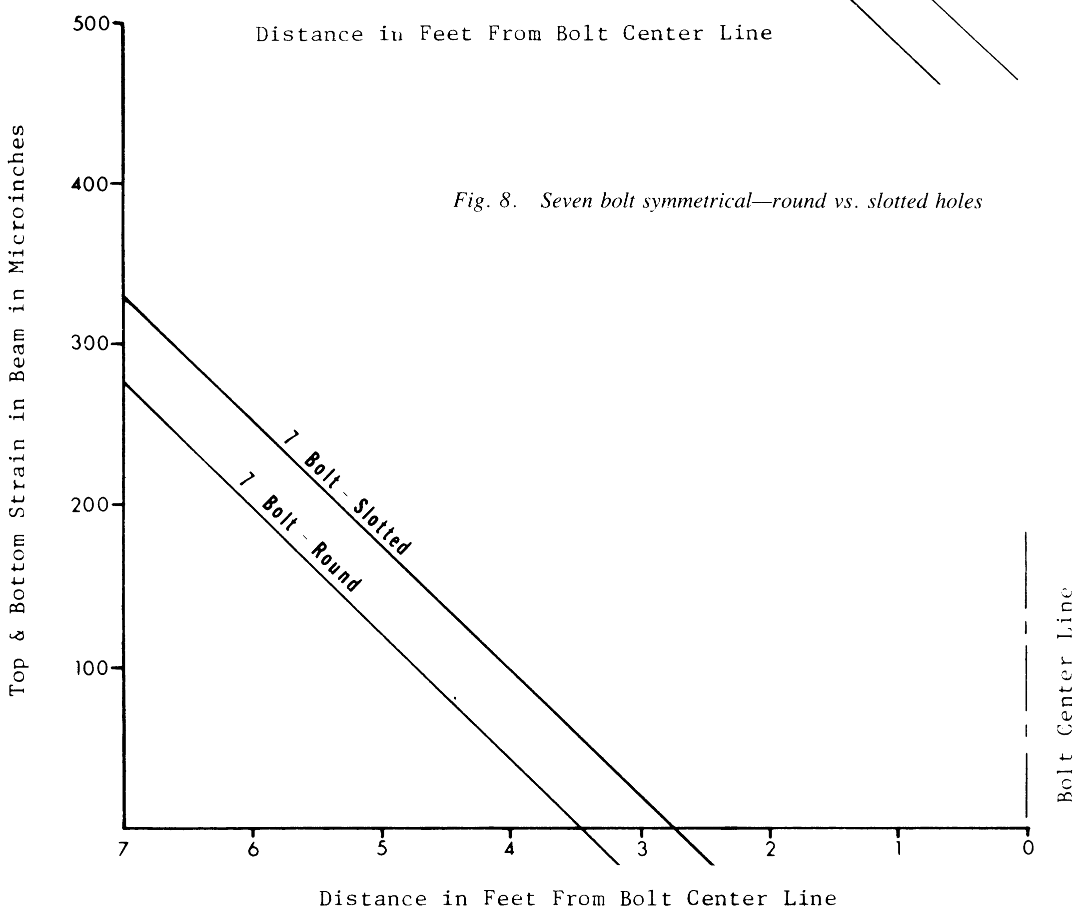


Fig. 8. Seven bolt symmetrical—round vs. slotted holes

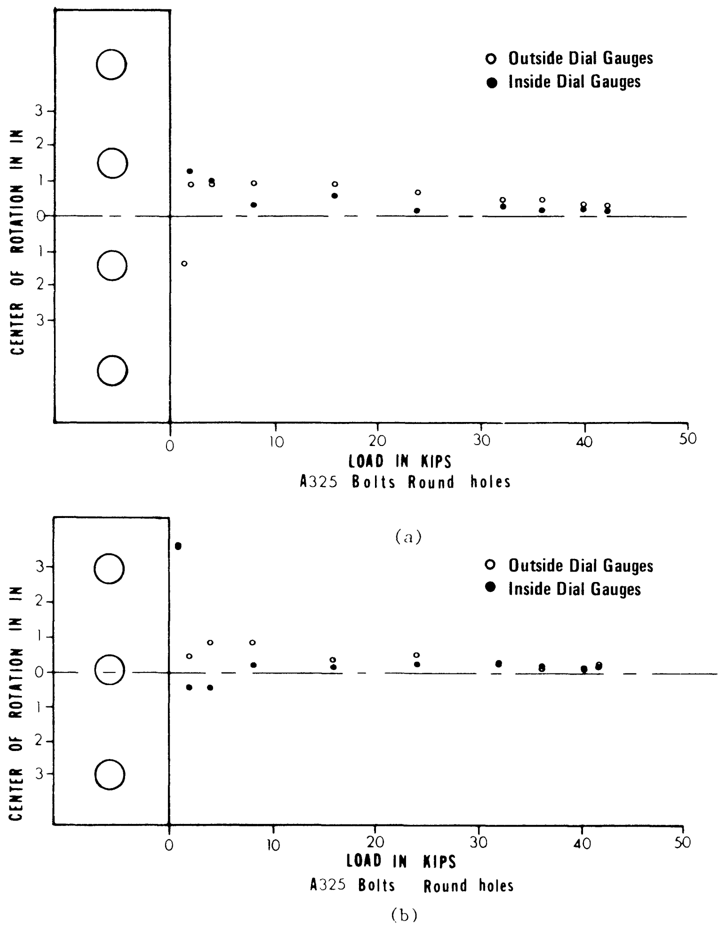


Fig. 9. Center of rotation off-axis A325 bolts round holes

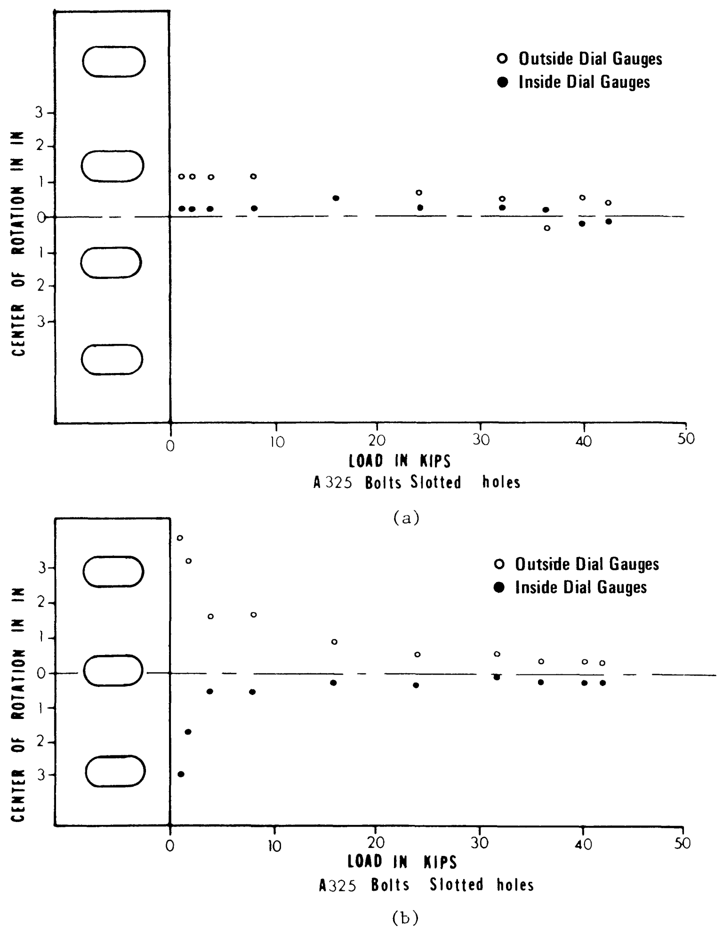


Fig. 10. Center of rotation off-axis A325 bolts slotted holes

indicated by the linear beam lines in Fig. 11. The plate connection, however, has a nonlinear moment rotation curve.[1] The actual end conditions will be those found at the intersection point of these two curves.

Beam Line Solution

The only effect of a higher beam yield stress on the beam line solution is to move the beam line out on both axes of the moment rotation diagram (Fig. 11). Since the connection moment-rotation curve is nearly flat where it intersects the beam lines, the moments for the two beams will be nearly equal. Equating these two moments, and noting that the end shear for the Gr. 50 steel beam is generally 50/36 times the end shear for the A36 beam, it follows that since

$$M_{36} \cong M_{50}$$

then

$$e_{36}\, V_{36} = e_{50}\, V_{50} = e_{50}\, V_{36} \left(\frac{50}{36}\right)$$

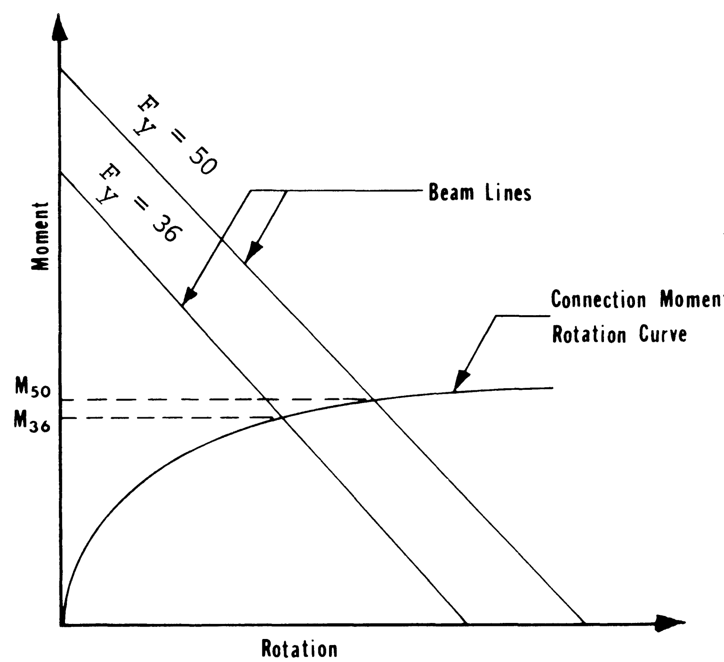


Fig. 11. Beam lines for $F_y = 36$ *and* $F_y = 50$ *steel beams*

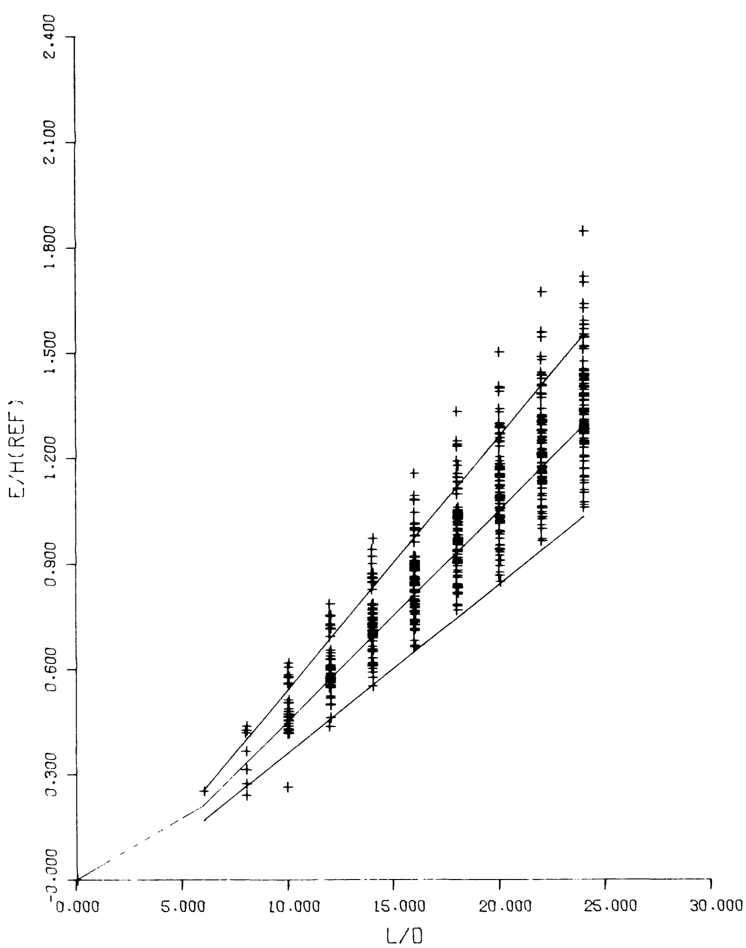


Fig. 12. Scatter plot symmetrical $F_y = 50$

so

$$e_{50} = e_{36}\left(\frac{36}{50}\right)$$

where M, e, and V are defined in Fig. 3. This relationship can be used as a modification factor for the design curve for calculating (e/h) of Ref. 1.

When using an A36 framing plate with a Gr. 50 steel beam the beam web thickness is 36/50 of the required framing plate thickness for the purpose of calculating the required d/t ratio to prevent bolt shear.

A modified version of program BEAMLINE[6] was used with the beam schedule in Table 2 to compare the beam line solution to the solution with the correction factor for about 1,000 beams. The middle line in Fig. 12 is the recommended $(e/h)_{ref}$ and the symbols are the beam line solutions. The outer lines are the $\pm$ 20% bounds. The number of points outside these bounds is slightly less for Gr. 50 steel beams than for A36 steel beams as reported by Richard.[1]

Table 2. Beam and Bolt Schedule for Design Curves

Beam	Section Modulus (in.^{3})	Bolt Diameter (in.)
W14x22	29.0	3/4
W16x26	38.3	3/4
W16x40	64.6	3/4
W18x35	57.9	3/4
W18x55	98.4	3/4
W21x44	81.6	3/4
W18x55	98.4	7/8
W21x44	81.6	7/8
W21x68	140.0	7/8
W24x76	176.0	7/8
W24x84	197.0	7/8
W24x94	221.0	7/8
W27x94	243.0	7/8
W24x94	221.0	1
W27x94	243.0	1
W30x116	329.0	1
W33x118	359.0	1
W33x141	448.0	1
W36x135	440.0	1
W36x182	662.0	1

L/d Limits

To ensure connection ductility by avoiding bolt shear and tension tearing of the plate or beam web, end rotations are limited to two thirds of the rotation that causes 0.3-in. deformation in the outer-most bolts. To satisfy this requirement for symmetrical (noncomposite) beams, the following restrictions on the *L/d* ratios are recommended:

Symmetrical Beams $F_y = 36$ ksi $\quad L/d \leq 36$
Symmetrical Beams $F_y = 50$ ksi $\quad L/d \leq 24$

Beams longer than these limits should be checked for excessive end rotation.

COMPOSITE BEAMS WITH SHORED CONSTRUCTION

An extensive analytical study was made with program BEAMLINE to develop design aids for single plate framing connections used with simply supported composite beams. Since full-scale beam tests of off-axis bolt groups resulted in essentially the same moment-rotation and center of rotation response as symmetrical connections, it is concluded that the behavior of the single plate is not affected by its location relative to beam's neutral axis. For shored construction then, the beam line solution does not differ from the beam line solution for symmetrical beams, except that the beam line itself must be located using the appropriate transformed section modulus.

Design Procedures

Using the beam and bolt schedules in Tables 3, 4 and 5, more than 5,000 designs were analyzed, resulting in the following recommended changes to the design procedure for noncomposite beams presented in Ref. 1:

Table 3. Composite Beam and Bolt Schedule

Beam	Slab Thickness (in.)	Section Modulus (in.^{3})	Bolt Diameter (in.)
W14x22	4	46.4	3/4
W16x26	4	59.5	3/4
W16x40	4	92.8	3/4
W18x35	4	86.2	3/4
W18x55	4	137	3/4
W21x44	4	120	3/4
W18x55	4.5	142	7/8
W21x44	4.5	124	7/8
W21x68	4.5	196	7/8
W24x76	4.5	241	7/8
W24x84	4.5	266	7/8
W24x94	4.5	298	7/8
W27x94	4.5	325	7/8
W24x94	5	307	1
W27x94	5	334	1
W30x116	5	443	1
W33x118	5	485	1
W33x141	5	587	1
W36x135	5.5	600	1
W36x160	5.5	719	1

Table 4. Composite Beam and Bolt Schedule Light Cover Plates

Beam	Slab Thickness (in.)	Plate Size (in.)	Section Modulus (in.^{3})	Bolt Diameter (in.)
W14x22	4	.5×4	75.6	3/4
W16x26	4	.5×4	96.3	3/4
W16x40	4	.5×6	142	3/4
W18x35	4	.5×5	131	3/4
W18x50	4	.5×6	178	3/4
W21x44	4	.5×5.5	176	3/4
W18x50	4.5	.5×6	184	7/8
W21x44	4.5	.5×5.5	182	7/8
W21x62	4.5	.5×7	252	7/8
W24x68	4.5	.5×8	308	7/8
W27x84	4.5	.5×9	405	7/8
W27x94	4.5	.5×9	441	7/8
W27x94	5	.5×9	452	7/8
W27x84	5	.5×9	415	1
W27x102	5	.5×9	480	1
W30x99	5	.5×9	503	1
W33x118	5	.5×10	642	1
W33x130	5	.5×10	695	1
W36x135	5.5	.5×10	770	1
W36x170	5.5	.5×10	933	1

Table 5. Composite Beam and Bolt Schedule Heavy Cover Plates

Beam	Slab Thickness (in.)	Plate Size (in.)	Section Modulus (in.^{3})	Bolt Diameter (in.)
W14x22	4	1×4	105	3/4
W16x26	4	1×4.5	133	3/4
W16x40	4	1.5×6	238	3/4
W18x35	4	1.5×5	220	3/4
W18x50	4	1.5×6	283	3/4
W21x44	4	1.5×5.5	287	3/4
W18x50	4.5	1.5×6	292	7/8
W21x44	4.5	1.5×5.5	295	7/8
W21x62	4.5	1.5×7	395	7/8
W24x68	4.5	1.5×8	492	7/8
W27x84	4.5	1.5×9	634	7/8
W27x94	4.5	1.5×9	669	7/8
W27x94	5	1.5×9	685	7/8
W27x84	5	1.5×9	649	1
W27x102	5	1.5×9	713	1
W30x99	5	1.5×9	758	1
W33x118	5	1.5×10	951	1
W33x130	5	1.5×10	1000	1
W36x135	5.5	1.5×10	1110	1
W36x170	5.5	1.5×10	1270	1

Case 1 — Shored Composite with No Cover Plates

A. Steel Stress Governs

Same as noncomposite beam except:

1. Use the beam depth, d, as defined in Fig. 13 to compute L/d.
2. Use the effective section modulus S_{eff} of the composite section in place of the steel beam section modulus, S, in the design equation for (e/h). S_{eff} is determined from Formula (1.11-1) of the AISC Specification.

B. Concrete Stress Governs

Same as above except:

1. *Use the effective section modulus of the top of the transformed section as:
   $S_{t\text{-}eff} = I_{eff}/(t + d - I_{eff}/S_{eff})$
   where
   t = thickness of concrete slab, in.
   d = depth of steel beam, in.
   I_{eff} from Formula (1.11-6) of the AISC Specification.
   S_{eff} from Formula (1.11-1) of the AISC Specification.
2. Since the concrete governs, do not use the $36/F_y$ correction factor for higher grade steel.

*For development of the equations in this section, see Ref. 8

Case 2 — Shored Composite with Cover Plates

A. Steel Stress Governs
Use the same procedure as for no cover plates, except multiply (e/h) by the additional term $(S_{eff\text{-}np}/S_{eff\text{-}cp})^{0.5}$ where
$S_{eff\text{-}np}$ = effective section modulus of the composite section with no cover plate.
$S_{eff\text{-}cp}$ = effective section modulus of the composite section with cover plate.

B. Concrete Stress Governs
Use the same procedure of Case 2-A, except:

1. *Use the effective section modulus of the top of the transformed section in the $(S_{t\text{-}eff\text{-}np}/S_{t\text{-}eff\text{-}cp})^{0.5}$ correction factor where
$S_{t\text{-}eff\text{-}np} = I_{eff\text{-}np}/[t+d-(I_{eff\text{-}np}/S_{eff\text{-}np})]$
$S_{t\text{-}eff\text{-}cp} = I_{eff\text{-}cp}/[t+d-(I_{eff\text{-}cp}S_{eff\text{-}cp})]$

2. Since the concrete governs, do not use the $36/F_y$ correction factor for the higher grade steels.

Summary of Design Curve

For noncomposite beams, or composite shored beams with or without cover plates, compute (e/h) from the following equation:

$$(e/h) = (e/h)_{ref}\left(\frac{n}{N}\right)\left(\frac{S_{ref}}{S_g}\right)^{0.4}\left(\frac{S_{gnp}}{S_g}\right)^{0.5}\frac{36}{F_{yg}}$$

where

$(e/h)_{ref}$ = $0.06\, L/d - 0.15$
L = beam length
d = beam depth as defined in Fig. 13
n = number of bolts
N = 5 for ¾-in. and ⅞-in. bolts, and 7 for 1-in. bolts
S_{ref} = 100 for ¾-in. bolts, 175 for ⅞-in. bolts, and 450 for 1-in. bolts
S_g = governing section modulus
S_{gnp} = governing section modulus with no cover plates
F_{yg} = governing minimum steel yield stress except for sections where concrete stress governs (F_{yg} = 36)

*For development of the equations in this section, see Ref. 8

L/d Limits

As with noncomposite beams, it is recommended that end rotations be limited to two-thirds of the rotation that causes 0.3 in. of deflection at the outermost bolt.

The Commentary to the AISC Specification Section 1.13.1[5] recommends a limit of $F_y/800$ on L/d for beams, this results in the following:

$L/d \leq 22$ for $F_y = 36$ ksi

$L/d \leq 16$ for $F_y = 50$ ksi

Although these limits are set to control deflections, they can be conservatively used to limit rotations also.

Support Conditions

This analysis is for composite beams that are simply supported. Slabs with negative reinforcing steel continuous over the supports constitute additional end restraint. Although the moment-rotation behavior of a single plate connection is independent of the beam section properties, it will be affected by these additional restraints.

The simply supported composite beam is the most critical case because single plate framing connections used with continuous slabs will have significantly less moment. In summary, the procedure presented here would be applicable for design of connections for simply sup-

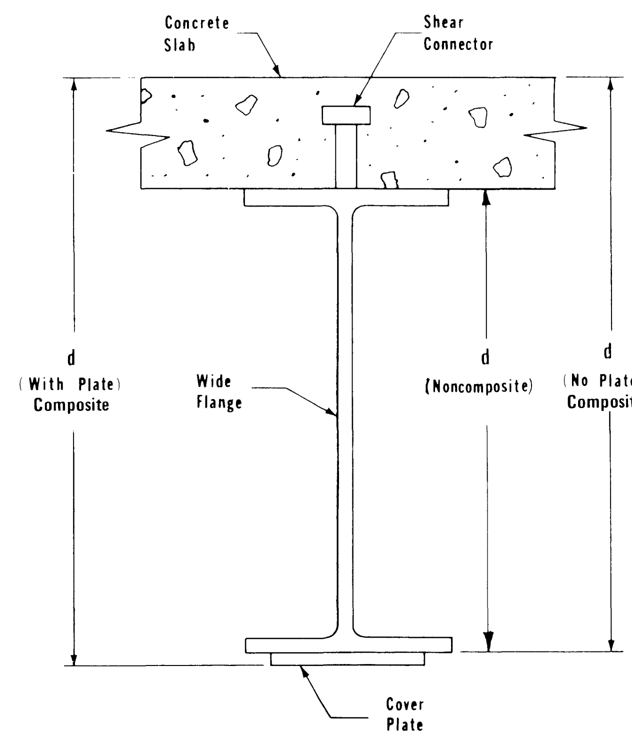


Fig. 13. Composite section

ported beams, beams supported by spandrel beams, beams at the edges of openings, or at columns (Fig. 14).

Results

In Fig. 15, typical results from the modified design curve are compared to the beam line solutions. As in Fig. 12, the middle line is the recommended $(e/h)_{ref}$ as a linear function of L/d and the outer lines are the $\pm 20\%$ bounds. The L/d ratios are limited on the high end to conform with the recommendations above and on the low end to one quarter of the span so that the beam lengths do not govern the maximum allowable effective flange widths. It was assumed that girder spacing did not govern the effective flange width.

Envelope errors and mean errors for the composite beams are comparable to those for noncomposite beams. The results are best for moderately proportioned designs. Very short beams and beams with deep bolt patterns tend to have larger errors.

Full Scale Beam Tests

The apparatus shown in Fig. 4 was used with a 50-ton jack to test the composite section shown in Fig. 16. The test procedure was the same as the noncomposite tests. The test schedule and results from ten tests are shown in Table 6. Tests 1 and 2 were identical in their setup and were run to establish repeatability of results.

Table 6. 3/4 in. ϕ A325 Bolts (Beam L/d = 14)

Case No.	No. of Bolts	Comments	Eccentricities (in.) Design Formula	Test No. 1	Test No. 2
1	5	3-in. pitch—concentric	17.2	20.9	21.6
2	4	3-in. pitch—eccentric	10.3	12.4	—
3	3	3-in. pitch—concentric	5.2	6.0	9.0
4	3	3-in. pitch—concentric beam shored when bolts torqued	5.2	6.7	—
5	3	6-in. pitch	10.3	16.2	19.2
6	3	3-in. pitch eccentric	5.2	5.6	8.3

Table 7. A307 Bolt Test in Composite Beam a = 3 in.

No. of Bolts	$(e + a)$ test	$(e + a)$ calculated
5	9.0	7.5
4	6.3	5.7
3	5.5	4.4
3 @ 6 in. oc	5.7	4.4
3 eccentric	6.2	4.4

Full scale composite beam tests were also run with A307 bolts. The results of these tests are compared in Table 7 to the results of the design curve reported by Richard:[7]

$$e = \frac{nh}{384}\frac{L}{d}$$

using the appropriate factor for concentrated load.[1]

Conclusion

The design procedures recommended by Richard (1980)[1] with the modifications recommended in this chapter may be used for composite beams. Appendix A contains examples of this procedure.

COMPOSITE BEAMS WITH UNSHORED CONSTRUCTION

In the derivations for shored construction, it was assumed that temporary shoring supported the composite beam until the concrete hardened. Under these circumstances the entire load is supported by the beam acting compositely. It is also common design practice to omit temporary shores. In this case, the dead load of the deck is supported by the rolled shape alone. Subsequent loads are resisted by the beam acting compositely with the slab. A beam line solution for unshored simply supported composite beams is derived here and the eccentricities calculated from it are compared to the design curve solutions.

Beam Line for Unshored Beams

Analysis of unshored composite beams requires segregating the load into the initial dead load of the rolled

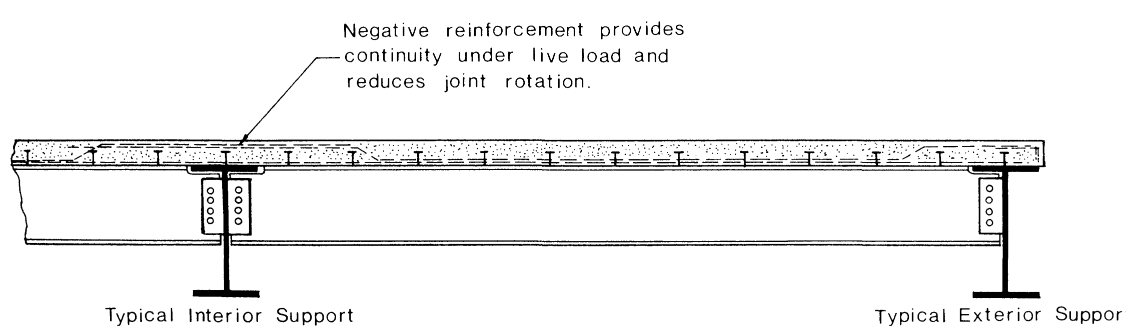


Fig. 14. Composite construction with single-plate framing connection

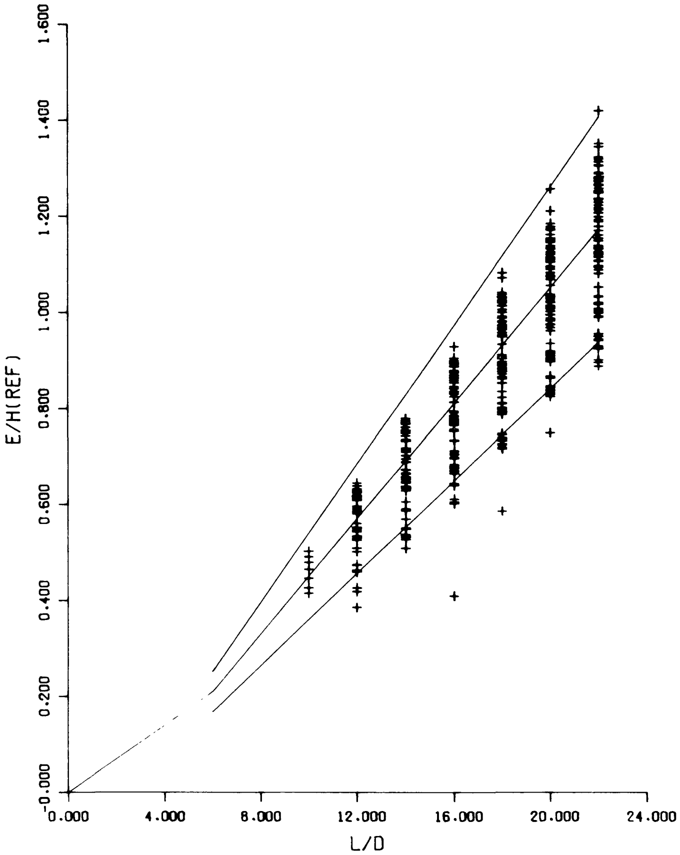


Fig. 15(a). Design curve with ±20% bound composite light plate F_y *= 36 ksi*

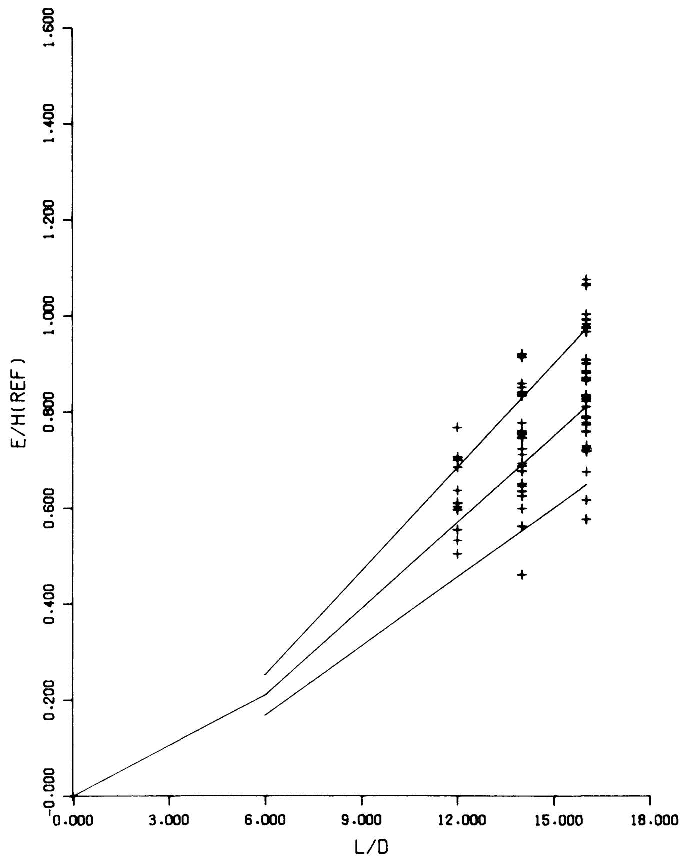
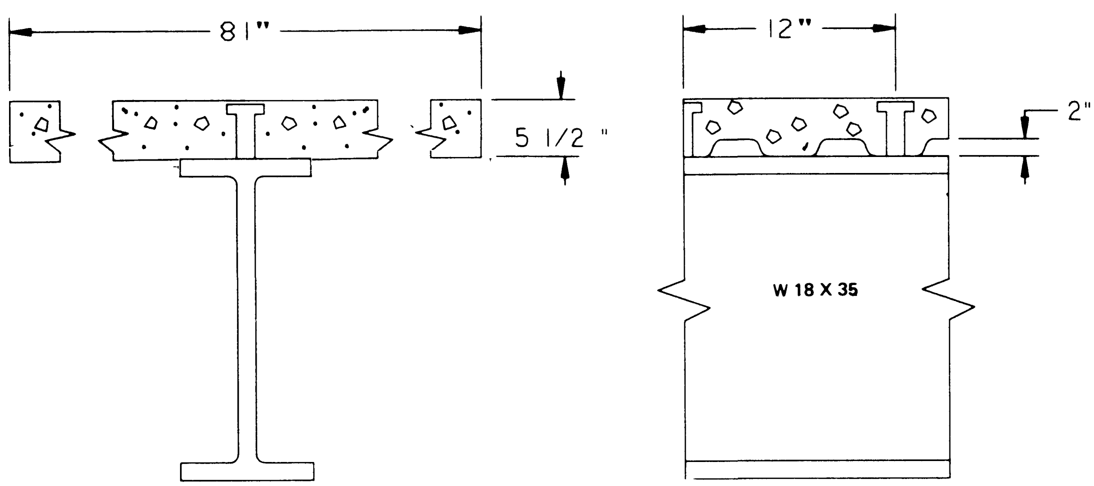


Fig. 15(b). Design curve with ± 20% bound composite light plate F_y *= 50 ksi*



Deck:
Inryco Steel Deck
1.5″ S(30″)
18 Ga.

Steel:
W18×35
f_y = 36 ksi
Length = 28′

Slab:
Thickness = 5″
Reinforcement:
WWF 6×6-6×6
1½″ from top of slab
Studs: ¾″ ϕ × 3½″
12″o.c. throughout

Concrete:
f'_c = 3 ksi
n = 9

Fig. 16. Composite beam section

shape and concrete slab, W_{idl}, and the additional superimposed load, W_l, required to take the composite beam to first yield. Under the initial dead load the bare beam and single framing plate rotate to point A, Fig. 17a. The beam line is defined by the fixed end moment, M'_{idl}, and the free end rotation, ϕ'_{idl}, due to the initial dead load on the rolled shape alone.

Since the connection moment-rotation curve is not a function of the section modulus, the connection continues to move along the same M-ϕ curve when the additional load, W_1, is applied, Fig. 17b. The additional rotation of the now composite beam can be no more than the free end rotation, ϕ'_1, caused by the additional load. Likewise, the additional moment cannot exceed the fixed end moment, M'_1. Since the composite beam is in the linear range up to first yield, the additional moment and rotation due to W_1 must vary linearly and be on the auxiliary beam line, Fig. 17b. Intersection point B is the solution.

An equivalent beam line can be constructed by extending the auxiliary beam line of Fig. 17b to the plate M-ϕ axes as shown in Fig. 18. Point B can then be located by the usual beam line method using the fixed end moment, M'_{eq}, and free end rotation, ϕ'_{eq}. M_{idl} and ϕ_{idl} are calculated from the initial dead load beam line. By similar triangles,

$$a = M_{idl}\,(\phi'_1/M'_1)$$
$$b = \phi_{idl}\,(M'_1/\phi'_1)$$

Thus,

$$M'_{eq} = M_{idl} + M'_1 + \phi_{idl}\,(M'_1/\phi'_1)$$
$$\mathrm{M}'_{idl} = m_{idl} + M'\left(1 + \frac{\phi_{idl}}{\phi'}\right)$$
$$\phi'_{eq} = \phi_{idl} + \phi' + M_{idl}\,(\phi'/M')$$
$$\phi'_{eq} = \phi_{idl} + \phi'\left(1 + \frac{M_{idl}}{M'}\right)$$

This derivation is for simply supported composite beams. As with the shored beams, continuity resulting from negative moment reinforcing steel in the slab over the supports constitutes additional constraint and reduces the connection moment.

Results

The beams of Table 8 were analyzed with a modified version of program BEAMLINE and the results compared to the design curve. Designs were limited to girder spacings greater than the maximum allowable effective flange width and less than one third of the span length. For comparison, all designs were analyzed for shored and unshored construction with live loads ranging from 50 psf to 100 psf. Typical results are shown in Figs. 19a and 19b.

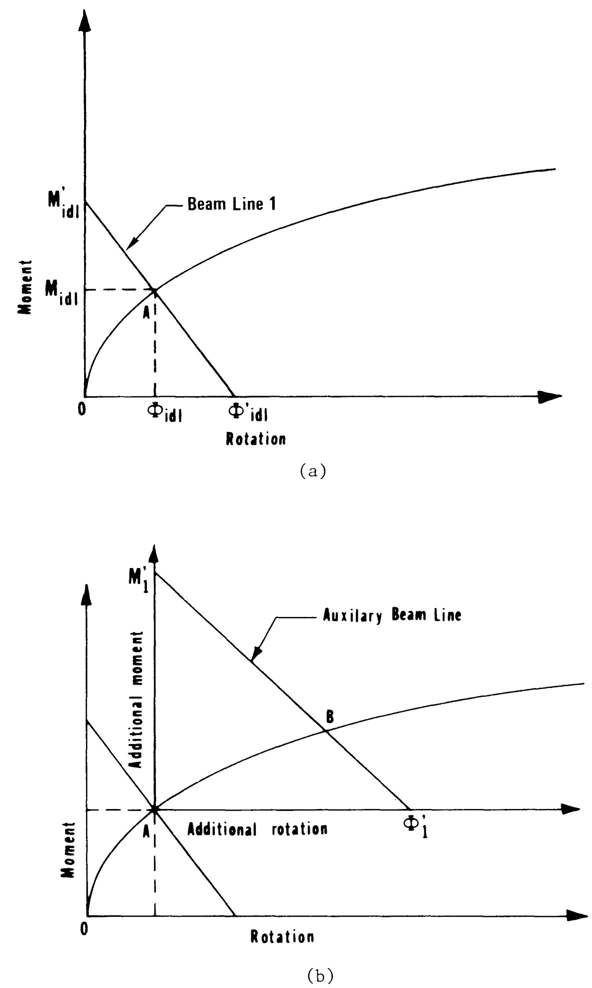


Fig. 17. Beam line for unshored construction

Conclusions

Unshored construction has two offsetting effects on connection moment. Part of the load is supported by the bare steel alone which moves the beam line out on the ϕ axis and increases the moment. On the other hand, the first yield load drops, which lowers the beam line on the ordinate and decreases the moment. Either of these phenomena can dominate. It is not possible to conclude unshored construction will always result in more or less connection moment than that for shored construction. However, the results of over 2,000 BEAMLINE analyses indicate the two construction techniques result in modest differences in connection moment and that the basic design curve still gives satisfactory results (Ref. 1). On the basis of studies conducted during this research, it is recommended the same design procedure be used for both shored and unshored construction.

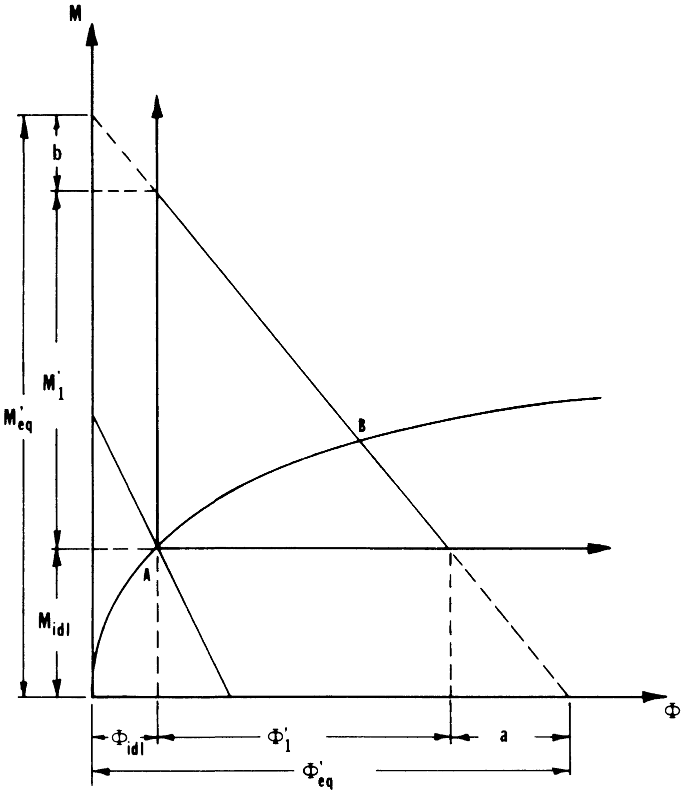


Fig. 18. Equivalent beam line

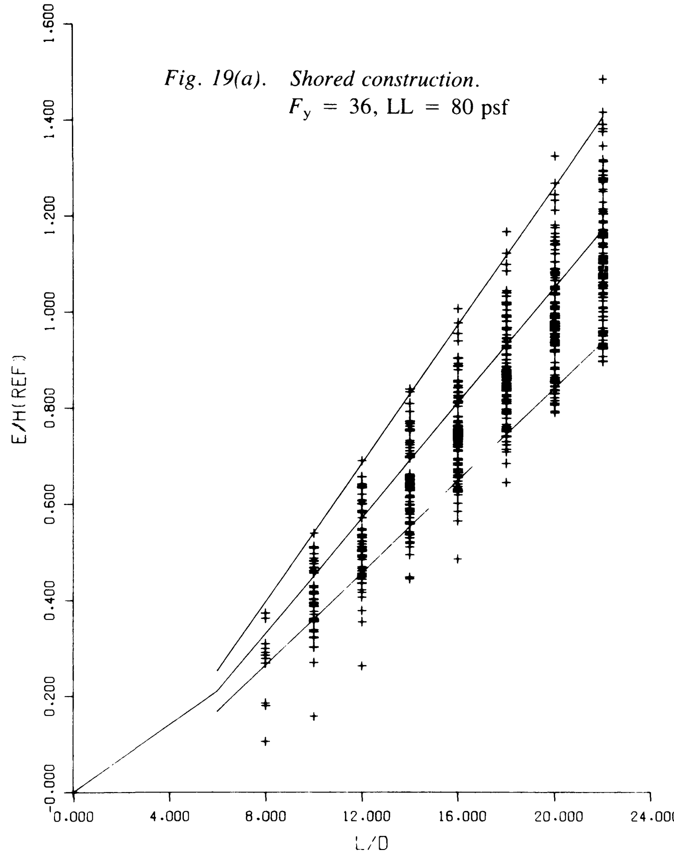


Fig. 19(a). Shored construction. F_y = 36, LL = 80 psf

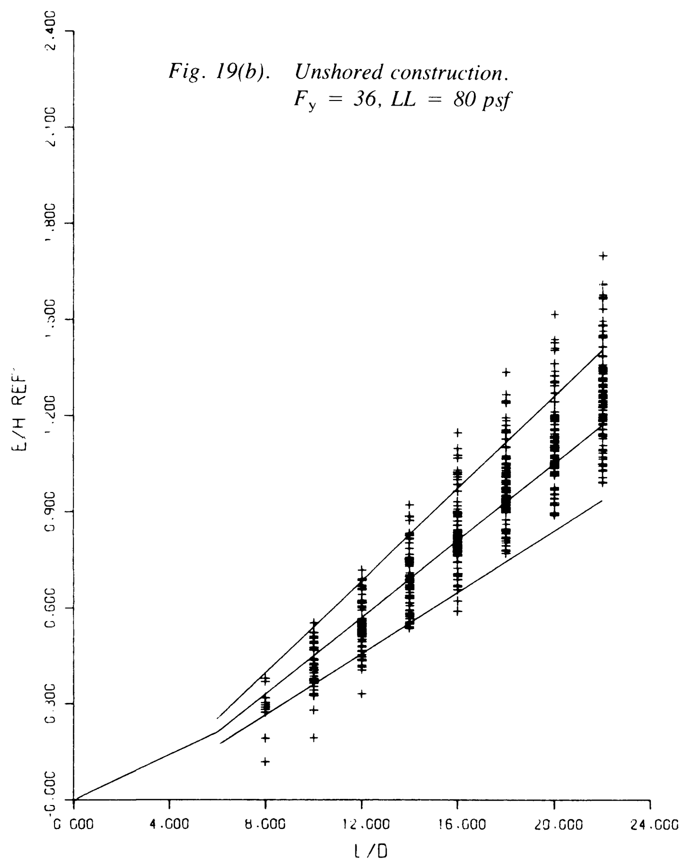


Fig. 19(b). Unshored construction. F_y = 36, LL = 80 psf

Table 8. Beam and Bolt Schedule for Shored vs. Unshored Composite Beam Studies

Beam	Slab Thickness (in.)	Bolt Diameter (in.)
W14x22	4	3/4
W16x26	4	3/4
W16x40	4	3/4
W18x35	4	3/4
W18x55	4	3/4
W21x44	4	3/4
W18x55	4.5	7/8
W21x44	4.5	7/8
W21x68	4.5	7/8
W24x76	4.5	7/8
W24x84	4.5	7/8
W24x94	4.5	7/8
W27x94	4.5	7/8
W24x94	5	1
W27x94	5	1
W30x116	5	1
W33x118	5	1
W33x141	5	1
W36x135	5.5	1
W36x160	5.5	1

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APPENDIX A

Design Example

Beam: W16×40, A36 steel with 4 in. Slab, S_{tr} = 92.8 in.^{3} Assume full composite action.

Span: 24 ft, simply supported

Loading: Uniform with W = 61.9 kips

Procedure

Step

1 Select A36 plate with $t_{plate} = 5/16$ in. ($t_{web} = 0.307$ in.)*

2 Try ¾ in. A325 bolts, $R = 61.9/2 = 30.9$ kips
$D/t = (3/4) / (5/16) = 2.4$
$N_{req'd} = 30.9/9.28 = 4$ bolts

3 $(e/h)_{ref} = 0.06\ l/d - 0.15 = 0.714$

$$(e/h) = 0.714 \times \left(\frac{4}{5}\right) \times \left(\frac{100}{92.8}\right)^{0.4} = 0.589$$

With p = 3 in., h = (4-1) × 3 = 9 in., and F_y = 36 ksi
$e = 0.589 \times 9 = 5.30$ in.

4 For a = 3 in., $V = R = 30.9$ kips
$M = 30.9 \times (5.30 + 3) = 256$ in.-kips

5 $$f_b = \frac{4 \times 256}{0.3125 \times 12^2} = 22.8 \text{ ksi} < 24 \text{ ksi}$$

$$f_v = \frac{30.9}{0.3125 \times 12} = 8.24 \text{ ksi} < 14.4 \text{ ksi}$$

6 $f_r = (22.8^2 + 8.24^2)^{1/2} = 24.2$ ksi
Use 70XX weld,

$$\text{req'd } D = \frac{24.2 \times .3125}{0.93} = 8.132/16^{ths}$$

Use 5/16 in. fillets each side

*For F_y = 50 ksi, minimum $t_w = t_{plate} \times \frac{36}{50}$