A Practical Design for Thin Composite Steel-Concrete Floor Systems

By Sompandh Wanant, P.E., M. ASCE

In recent years, it has become more desirable, and in many cases necessary, for architects and engineers to design buildings and structural frames with beams and girders of limited depth. Shallower structural depth allows building floor-to-floor height to be lowered and the amount of materials used – such as exterior cladding, interior walls, partitions, and stairs – to be reduced. In high-rise building construction, it allows extra floors to be added within the proposed building height. On expansion projects, a shallower structural depth helps facilitate the need to match the existing floor elevations.

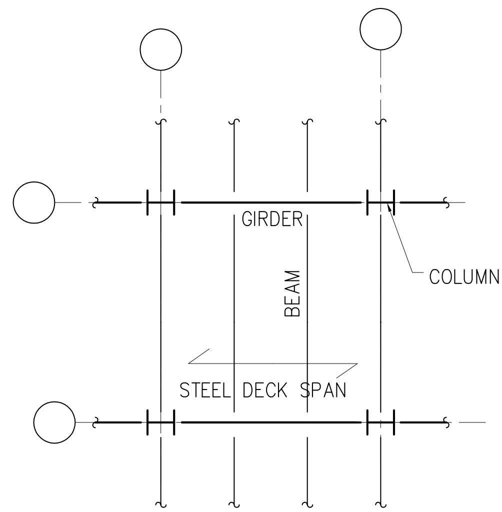

Figure 1: Framing plan.

Practical Design

This system has been developed for use in building floor construction, specifically for typical rectangular or square column bay areas of around 1,000 square feet. One of the most economical and widely used floor framing systems is the third-point loading of two in-fill beams on girders which span between columns (*Figure 1*). Conventional composite steel-concrete floor framing consists of rather deep steel beams and girders which provide the most cost-effective design in terms of tonnage of structural steel used. In this article, American Institute of Steel Construction (AISC)-standard structural steel shapes will be used for both composite beams and girders, and will be made as shallow as practically possible. It should be noted that moderate column bay sizes are used in the design examples; however, larger column bays in the range of 30 feet × 45 feet can be designed economically.

Although this system is intended for building floor construction, the concept may be applied to any other composite beams/girders requiring a shallower depth.

The Beam

The conventional composite beam, consisting of normal or light weight concrete, composite steel deck and a steel beam, is made shallower by replacing the steel beam section with a shallower (heavier) one. The concrete thickness over the steel deck is between 2½ inches to 4½ inches, and the standard 1½, 2 or 3-inch deep steel deck may be used as required. Since the total depth of the composite girder is the depth of the system, the total design depth of the composite steel beam should be made as deep as possible but not greater than the total depth of the composite girder (*Figure 2*). It should be noted that, if desired, square or rectangular steel tubes (HSS sections) may be used in lieu of wide flange shapes and a cover plate at the bottom of the steel section may be added.

The Girder

The composite steel-concrete section consists of an inverted structural tee (WT), a steel plate and a standard steel-concrete composite slab (*Figure 3, page 16*). For the system to be practical, flexible and efficient, the

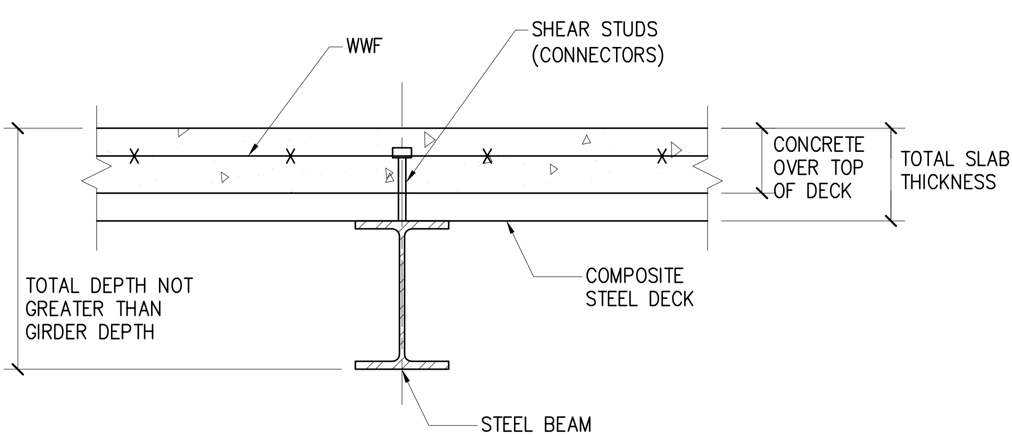

Figure 2: Typical beam section.

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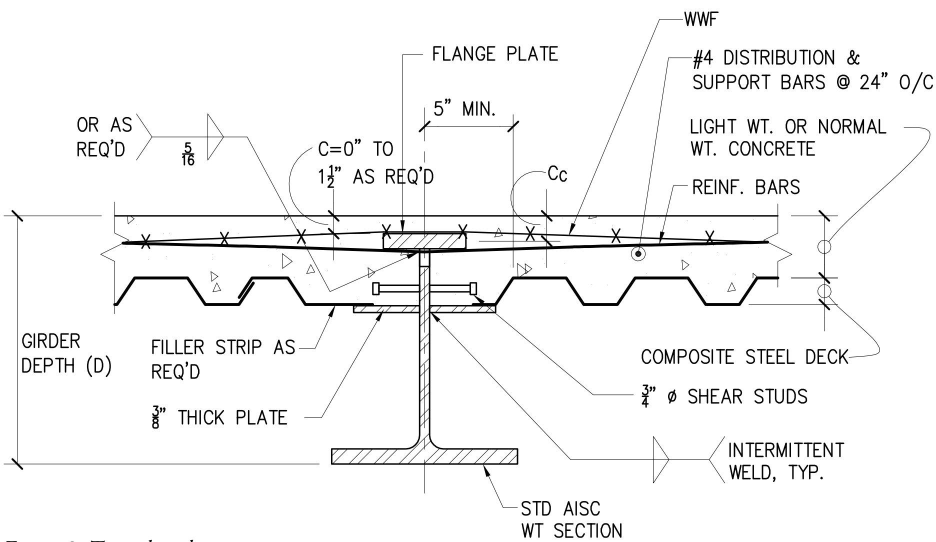

Figure 3: Typical girder section.

standard structural tee (WT) and a thick and narrow flange plate are selected. The thick and narrow flange plate provides not only needed strength and minimizes creep and long-term deflection, but also leaves adequate space required for field attachment of the steel deck to the steel girder built-up section. The steel girder built-up section and the steel-concrete composite slab form a very strong T-section flexural member. One major advantage of this composite section is that field welding of the shear connectors to the steel girder top flange is not required since the top flange is embedded in the concrete slab with the top of the flange at or near the top of the concrete slab.

The strength and stiffness of the composite girder can be calculated by transforming the concrete portion into steel using Modular Ratio, $n = E_s/E_c$; $E_c = w_c^{3/2}(33)\ (f'_c)^{1/2}$. The imaginary homogeneous transformed section is analyzed for its physical properties by the elastic methods of analysis.

As in any T-beam section, substantial flexural horizontal shear exists at the intersection of the web and the flange. This horizontal shear can be determined using the familiar equation, $v = VQ/I$. For thin concrete slabs above the top of the steel deck, reinforcement is usually required to increase the shear capacity. Shear-friction provisions given in American Concrete Institute (ACI) 318 Section 11.7 and its commentary are referred to for the design of concrete slab shear reinforcement. *Figure 4* shows the concrete slab critical shear planes/surfaces. Slippage at the interface of the concrete and WT web can also occur; steel stud shear connectors are used to prevent slippage at the web. The number of shear studs required can be determined by calculating the difference between the shear capacity, found using ACI 318 Equation (11-25): $V_n = A_{vf} f_y \mu$, and the total design shear at the web. For other critical shear planes ACI 318 (Commentary Section R11.7.3) Equation: $V_n = 0.8\ A_{vf}\ f_y + A_c\ K_1$ is employed to obtain the amount of shear reinforcement. Please refer to ACI 318 for more design information.

With the existence of the steel deck, one may want to consider its strength, along with concrete, to help resist the horizontal shear. Additional shear resistance of the steel deck (limited to the attachment to the support or the side lap capacity) is beyond the scope of this article. In the design examples, the concrete below the top of the steel deck, the steel deck and the steel plate supporting the deck are neglected from calculations; together they constitute a few percentage points of the total stiffness.

Design Criteria for the Steel-Only Asymmetric Built-Up Section

To ensure proper and uniform design of the composite girder section and to set the lower strength limit of the steel-only built-up section, the built-up section must have adequate strength to support the total slab dead load and its supporting steel frames.

Serviceability Requirements

AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360-05) section L5 requires that "The effect of vibration on the comfort of the occupants and the function of the structure shall be considered." As in any floor system design, the vibration characteristics of the floor system (i.e. the natural frequencies, the amplitude/acceleration due to the certain appropriate dynamic loading) must be evaluated and satisfied. Refer to AISC *Steel Design Guide* #11, "Floor Vibrations Due to Human Activity", for design information and procedure.

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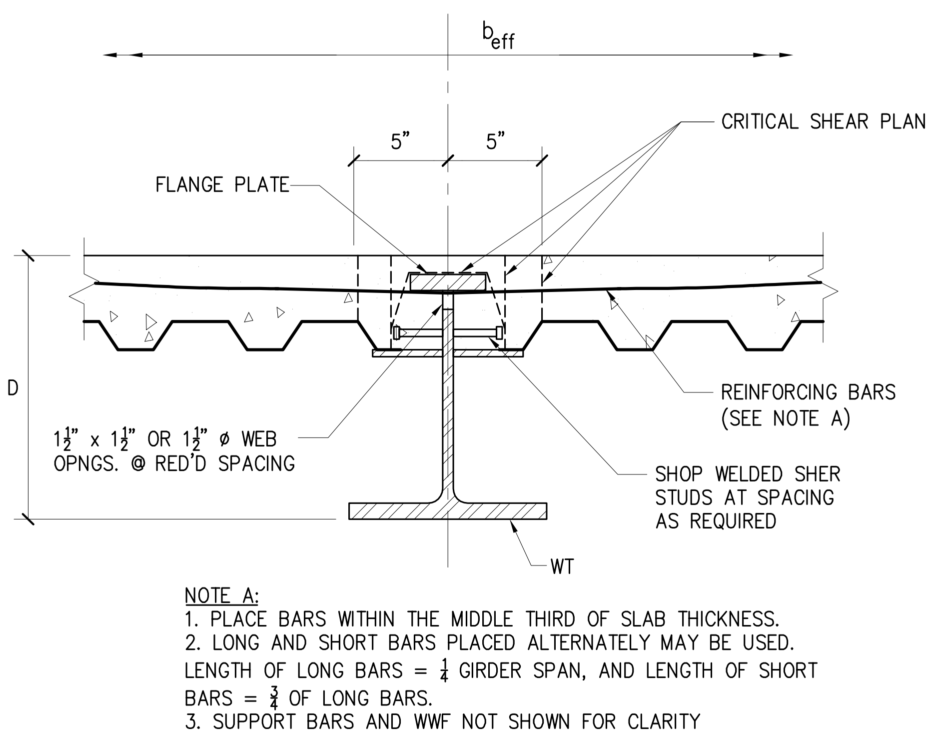

Figure 4: Critical shear planes.

Connections and Choice of Steel Girder Fabrications

The system uses standard AISC beam-to-girder and girder-to-column connections, so the detailed connections are not shown. In the examples, the author used the actual girder span in the design calculations, i.e. the column line dimension minus two times the distance between the column center line and the shear plate bolt holes. For a heavy girder-to-column connection where the depth of the girder web is limited, a beam seat may be required at the bottom of the girder; i.e. a standard AISC seated beam connection may be used.

Since it is necessary to provide web openings or drilled holes for the reinforcing bars, and the WT section is cut from a large W section, it may be easier and more cost effective to fabricate the required WT, with the notches for web openings, in a similar manner to fabricating the castellated beam. If this method is chosen, the actual total depth of the WT should be used in the calculations. Alternately, one may prefer to simply make the drilled holes. It should be noted that if asymmetric shape rolled sections are available domestically the fabrication cost will be reduced.

Advantages over Other Thin Floor Systems

At least a couple of thin/shallow floor systems, such as the Girder-Slab and Slimflor systems, are available for use in North America and Europe. Both systems can be used in several types of buildings, but are best suited for certain applications. The advantages of the proposed thin floor system over these systems are for general applications only:

- The proposed thin floor system is much lighter than both the Girder-Slab and the Slimflor systems.
- The composite beam of the proposed thin floor system can span much longer than 30 feet, whereas the Slimflor slab has the span limit of less than 30 feet.
- Due to lighter slab weight, both the beam and the girder can span longer distances resulting in larger open floor spaces.
- The space between beams and girders may be utilized as rectangular or square recessed ceiling space, or as additional utility space.
- The system is based on the conventional composite floor system, therefore it has all the benefits of composite floor

Design Examples

Example 1

Design a typical interior composite girder for an office floor space. The depth of the girder shall be as shallow as practically possible; provide shoring as required. W12 columns may be used.

Given:

Floor framing as shown in *Figure 1* with column line dimensions of 28 ft × 28 ft
Beam span = 28 ft, Beam spacing = 28/3 = 9.33 ft
Girder span = 28.0 – 2(0.50 + 0.25) = 26.50 ft (Girder frames to W12 column flange)
Concrete: 3.25 in, 110 pcf light weight concrete, f'_c = 4,000 psi
Composite steel deck: 19 gage, 1.50-inch deep; Structural steel: F_y = 50 ksi
Live load = 50 psf; Partition load = 15 psf; Mechanical, electrical and misc. = 5 psf

Solution:

Dead load: Concrete and steel deck = 40 psf; Total dead load = (40 + 15 + 5) = 60 psf
Assumed beam weight = 40 plf
Live load: Live load reduction, $L = L_0 (0.25 + 15 / (K_{LL} A_T)^{1/2})$ (IBC Section 1607.9.1)
Live load, $L = 50 (0.25 + 15 / (2 \times 9.333 \times 28)^{1/2}) = 50 (0.906) = 45.3$ psf
Assumed girder weight = 75 plf

Loads on girder:

Dead load, $P_{DL} = (60 \times 9.333 + 40) \times 28 = 16{,}800$ lbs
Live Load, $P_{LL} = 45.3 \times 9.333 \times 28 = 11{,}838$ lbs
Moment: $M_{DL} = 16.800 (8.58) + (0.075) (26.5)^2/8 = 150.7$ kip-ft
$M_{LL} = 11.838 (8.58) = 101.7$ kip-ft
$M_{TL} = 150.7 + 101.7 = 252.4$ kip-ft
Reaction: $R_{DL} = 16.80 + (0.075) (26.50 / 2) = 17.79$ kips
$R_{LL} = 11.84$ kips
$R_{TL} = 17.79 + 11.84 = 29.63$ kips
Section modulus: $F_b = M/S$, $S_{req'd} = 252.4 \times 12/33.0 = 91.8\ in^3$
Moment of inertia and control of deflection, (E = 29,000 ksi):
$\Delta_{max} = Pa (3L^2 – 4a^2)/24 EI$ (Two equal point loads)
$\Delta_{max} = 5wL^4/384EI$ (Uniform load)
$\Delta_{DL} = 648.8 / I + 28.7 / I = 677.5 / I$
$\Delta_{LL} = 457.2 / I$
$\Delta_{TL} = 1134.7 / I$

From the above design parameters, required girder section properties: $S_{req'd} = 91.8\ in^3$, $I_{req'd} = 856.4\ in^4$ (Use deflection limits, $\Delta_{LL} = L / 360$ or $\Delta_{TL} = L / 240$).

Design of composite girder:

Refer to *Figure 4*, use 1 inch of concrete cover over the flange plate, try WT 10.5 × 55.5 with 1 in × 4 in flange plate, total girder depth = 12.755 in, effective flange width, b_{eff} = 26.50 × 12 / 4 = 79.50 in; using the elastic methods of analysis for transformed section, one can find: $S_{tr} = 125.2\ in^3$ and $I_{tr} = 960.8\ in^4$ (OK). Using Equation v = VQ / I, to find shear stresses at critical locations (*Figure 4*), and ACI 318 Section 11.7 one can find: Slab shear reinforcement: #4 @ 8 inches; and shear studs: ¾ in diameter @ 15 inch. See detailed calculations below:

Horizontal shear at girder center line and center of the slab (flange)
$v = VQ / I$
$Q = (21.53) (12.755 – 1.625 – 7.674) = 74.41\ in^3$
$v = 29.63 \times 74.41 / 960.8 = 2.295$ kip/in

Shear stress varies linearly from maximum at girder center line to zero at the end of the flange, from *Figure 4*, at distance 5 inches from center line girder where the slab thickness is 3.25 inches, the shear stress
$v_1 = (2.295 / 2) (1 / 3.25) (79.50 / 2 – 5.000) / (79.50 / 2) = 0.309$ ksi

Using Load Factor, LF = (1.2 × 67 + 1.6 × 45.3) / (67 + 45.3) = 1.36
$v_{1u} = 1.36 \times 309 = 420$ psi
$v_{1n} = 420 / 0.75 = 560$ psi (Reduction Factor, ø = 0.75)
$V_{1n} = 560\ A_c$ (A_c is area of concrete section resisting the shear)
< 800 A_c OK (ACI 318 § 11.7.5, Vn not greater than 0.2 $f'_c A_c$ nor 800 A_c)

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From ACI Commentary Section R11.7.3

$V_n = 0.8\ A_{vf}\ f_y + A_c\ K_1$

$560\ (3.25 \times 12) = 0.8\ A_{vf}\ (60{,}000) + (3.25 \times 12)\ (200)$

$A_{vf} = 0.29\ in^2$

By inspection, reinforcement at this location governs the design; use #4 @ 8", As = 0.30 in^2.

Shear studs required to prevent slippage at WT web:

Design shear, v = 2.295 / 2 kip/in

Total V_n = (2.295 / 2) (12) (1.36 / 0.75) = 24.97 kips

Capacity of shear-friction reinforcement, $V_n = A_{vf}\ f_y\ \mu$ = 0.30 (60) (0.525) = 9.45 kips (ACI 318 § 11.7.4.3, $\mu = 0.70 \times 0.75 = 0.525$)

Shear capacity required by shear studs = 24.97 – 9.45 = 15.52 kips (Ultimate capacity)

Allowable shear capacity required = 15.52 × 0.75 / 1.36 = 8.56 kips

For ¾-inch diameter shear stud, allowable shear = 13.30 × 0.83 = 11.04 kips, shear studs spacing = 12 × 11.04 / 8.56 = 15.47 in. Use ¾-inch diameter shear studs @ 15 inches.

Example 2

Design a typical interior composite girder for an office floor with the column bay spacing of 30 ft × 30 ft, and design live load of 80 psf throughout for the flexibility of future corridor arrangements.

Given:

Floor framing as shown in *Figure 1* with column line dimensions of 30 ft × 30 ft

Beam span = 30 ft, Beam spacing = 10 ft

Girder span = 30.0 – 2(0.50 + 0.25) = 28.50 ft (Girder frames to W12 column flange)

Concrete: 3.50 in, 145 pcf normal weight concrete, f'_c = 4,000 psi

Composite steel deck: 20 gage, 2-inch deep; Structural steel: F_y = 50 ksi

Live load = 80 psf; Partition load = 15 psf; Mechanical, electrical and misc. = 5 psf

Solution:

Select 1.50-inch concrete cover over the flange plate (for fire protection), try WT 12 × 73 with 1 in × 4.50 in flange plate, total girder depth = 14.87 in. Following the same design procedure as in Example 1, one can find: Composite Girder, I_{tr} = 1849.6 in^4 ($I_{req'd}$ = 1555.0 in^4), S_{tr} = 193.6 in^3 ($S_{req'd}$ = 151.4 in^3). Slab shear reinforcement: (2)#4 @ 12 inch (Spacing ~ 3 times slab thickness, OK). Shear studs: ¾ in diameter @ 14 inches.

Note on Total Building Cost

From Example 2, the steel girder weight is 88.3 plf (15.3 plf for flange plate plus 73 plf for WT). As the estimated beam size is W 8 × 35 (not shown in the example), we calculate the amount of steel to be 6.44 psf of the floor area. By determining the amount of steel required for a conventional composite girder and beam design (W 18 × 60 for girder and W 14 × 26 for beam), we get 4.60 psf. Since the size of the steel columns and lateral bracing members varies greatly, from less than 1 psf to more than 1.5 psf, due to many parameters, let's assume that the combined weight of columns and bracing members is approximately 1.25 psf. From the above information, we figure the weight increase of the steel frames to be (6.44 + 1.25) / (4.60 + 1.25) = 1.315, or a 31.5 % increase. According to many publications, including AISC's, the cost of raw material is only about one third of the total cost of the structural steel frames, the rest being the cost of fabrication and erection; and, the cost of the steel frames is about 10 % to 12 % of the total building cost. Therefore, we can conclude that the increase in total building cost is approximately (31.5 / 3) × 0.11 % = 1.16 %. Since the increase in total building cost due to steel weight is very small, it is believed that, in most cases, the saving in cost on the exterior wall (a major cost) and all other interior constructions can adequately offset the additional cost of steel.

construction, including foundations, seismic load resisting systems; other benefits are floor framing flexibility and lower overall costs.

Use of Computer Programs and Laboratory Testing

As shown in the examples, the calculation for strength and stiffness is straightforward, but for everyday design tasks, a computer program will enhance and facilitate the design process. The structural design of the composite girder presented herein should be laboratory tested to confirm the theoretical and design analysis results and to make further design developments should one wish to do so.

Conclusions

The above design examples show that a practical thin floor system can be designed and constructed. In both examples, approximately 7 to 9 inches in total girder depth can be saved for most commonly used beam and girder spans. The system utilizes all the elements of conventional composite steel-concrete construction, i.e. normal or light weight concrete, a composite steel deck, welded shear studs, standard structural steel shapes and typical steel-to-steel connections. Though the system generally requires shoring of the beams and girders due to their shallow sections, a significant amount of field welding of shear studs to the girder is not required. Unlike other thin floor systems, this system maintains the use of a thin composite steel-concrete deck and standard structural steel shapes; therefore, the system will be as cost effective and flexible for application as the conventional composite floor construction.▪

Acknowledgments

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