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Stub Girder Floor Systems

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18.1 Introduction

The stub girder system was developed in response to a need for new and innovative construction techniques that could be applied to certain parts of all multi-story steel-framed buildings. Originated in the early 1970s, the design concept aimed at providing construction economies through the integration of the electrical and mechanical service ducts into the part of the building volume that is occupied by the floor framing system [11, 12]. It was noted that the overall height of the floor system at times could be large, leading to significant increases in the overall height of the structure, and hence the steel tonnage for the project. At other times the height could be reduced, but only at the expense of having sizeable web penetrations for the ductwork to pass through. This solution was often accompanied by having to reinforce the web openings by stiffeners, increasing the construction cost even further.

The composite stub girder floor system subsequently was developed. Making extensive use of relatively simple shop fabrication techniques, basic elements with limited fabrication needs, simple connections between the main floor system elements and the structural columns, and composite action between the concrete floor slab and the steel load-carrying members, a floor system of significant strength, stiffness, and ductility was devised. This led to a reduction in the amount of structural steel that traditionally had been needed for the floor framing. When coupled with the use of continuous, composite transverse floor beams and the shorter erection time that was needed for the stub girder system, this yielded attractive cost savings.

Since its introduction, the stub girder floor system has been used for a variety of steel-framed buildings in the U.S., Canada, and Mexico, ranging in height from 2 to 72 stories. Despite this relatively widespread usage, the analysis techniques and design criteria remain unknown to many designers. This chapter will offer examples of practical uses of the system, together with recommendations for suitable design and performance criteria.

18.2 Description of the Stub Girder Floor System

The main element of the system is a special girder, fabricated from standard hot-rolled wide-flange shapes, that serves as the primary framing element of the floor. Hot-rolled wide-flange shapes are also used as transverse floor beams, running in a direction perpendicular to the main girders. The girder and the beams are usually designed for composite action, although the system does not rely on having composite floor beams, and the latter are normally analyzed as continuous beams. As a result, the transverse floor beams normally use a smaller drop-in span within the positive moment region. This results in further economies for the floor beam design, since it takes advantage of continuous beam action.

Allowable stress design (ASD) or load and resistance factor design (LRFD) criteria are equally applicable for the design of stub girders, although LRFD is preferable, since it gives lower steel weights and simple connections. The costs that are associated with an LRFD-designed stub girder therefore tend to be lower.

Figure 18.1 shows the elevation of a typical stub girder. It is noted that the girder that is shown

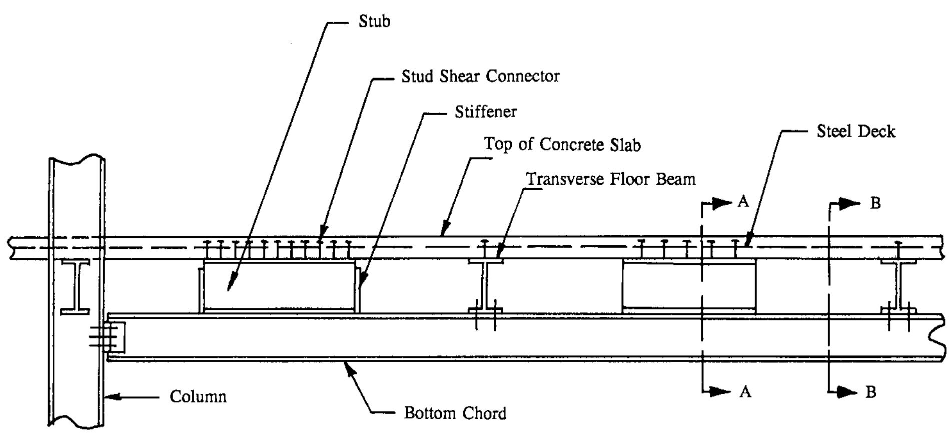

FIGURE 18.1: Elevation of a typical stub girder (one half of span is shown).

makes use of four stubs, oriented symmetrically with respect to the midspan of the member. The locations of the transverse floor beams are assumed to be the quarter points of the span, and the supports are simple. In practice many variations of this layout are used, to the extent that the girders may utilize any number of stubs. However, three to five stubs is the most common choice. The locations of the stubs may differ significantly from the symmetrical case, and the exterior (= end) stubs may have been placed at the very ends of the bottom chord. However, this is not difficult to address in the modeling of the girder, and the essential requirements are that the forces that develop as a result of the choice of girder geometry be accounted for in the design of the girder components and the adjacent structure. These actual forces are used in the design of the various elements, as distinguished from the simplified models that are currently used for many structural components.

The choices of elements, etc., are at the discretion of the design team, and depend on the service requirements of the building as seen from the architectural, structural, mechanical, and electrical viewpoints. Unique design considerations must be made by the structural engineer, for example, if it is decided to eliminate the exterior openings and connect the stubs to the columns in addition to the chord and the slab.

Figure 18.1 shows the main components of the stub girder, as follows:

1. Bottom chord
2. Exterior and interior stubs
3. Transverse floor beams
4. Formed steel deck
5. Concrete slab with longitudinal and transverse reinforcement
6. Stud shear connectors
7. Stub stiffeners
8. Beam-to-column connection

The bottom chord should preferably be a hot-rolled wide-flange shape of column-type proportions, most often in the W12 to W14 series of wide-flange shapes. Other chord cross-sections have been considered [19]; for example, T shapes and rectangular tubes have certain advantages as far as welded attachments and fire protection are concerned, respectively. However, these other shapes also have significant drawbacks. The rolled tube, for example, cannot accommodate the shear stresses that develop in certain regions of the bottom chord. Rather than using a T or a tube, therefore, a smaller W shape (in the W10 series, for example) is most likely the better choice under these conditions.

The steel grade for the bottom chord, in particular, is important, since several of the governing regions of the girder are located within this member, and tension is the primary stress resultant. It is therefore possible to take advantage of higher strength steels, and 50-ksi-yield stress steel has typically been the choice, although 65-ksi steel would be acceptable as well.

The floor beams and the stubs are mostly of the same size W shape, and are normally selected from the W16 and W18 series of shapes. This is directly influenced by the size(s) of the HVAC ducts that are to be used, and input from the mechanical engineer is essential at this stage. Although it is not strictly necessary that the floor beams and the stubs use identical shapes, it avoids a number of problems if such a choice is made. At the very least, these two components of the floor system should have the same height.

The concrete slab and the steel deck constitute the top chord of the stub girder. It is made either from lightweight or normal weight concrete, although if the former is available, even at a modest cost premium, it is preferred. The reason is the lower dead load of the floor, especially since the shores that will be used are strongly influenced by the concrete weight. Further, the shores must support several stories before they can be removed. In other words, the stub girders must be designed for shored construction, since the girder requires the slab to complete the system. In addition, the bending rigidity of the girder is substantial, and a major fraction is contributed by the bottom chord. The reduction in slab stiffness that is prompted by the lower value of the modulus of elasticity for the lightweight concrete is therefore not as important as it may be for other types of composite bending members.

Concrete strengths of 3000 to 4000 psi are most common, although the choice also depends on the limit state of the stud shear connectors. Apart from certain long-span girders, some local regions in the slab, and the desired mode of behavior of the slab-to-stub connection (which limits the maximum f_c' value that can be used), the strength of the stub girder is not controlled by the concrete. Consequently, there is little that can gained by using high-strength concrete.

The steel deck should be of the composite type, and a number of manufacturers produce suitable types. Normal deck heights are 2 and 3 in., but most floors are designed for the 3-in. deck. The deck ribs are run parallel to the longitudinal axis of the girder, since this gives better deck support on the transverse floor beams. It also increases the top chord area, which lends additional stiffness to a member that can span substantial distances. Finally, the parallel orientation provides a continuous rib trough directly above the girder centerline, improving the composite interaction of the slab and the girder.

Due to fire protection requirements, the thickness of the concrete cover over the top of the deck ribs is either 4-3/16 in. (normal weight concrete) or 3-1/4 in. (lightweight concrete). This eliminates the need for applying fire protective material to the underside of the steel deck.

Stud shear connectors are distributed uniformly along the length of the exterior and interior stubs, as well as on the floor beams. The number of connectors is determined on the basis of the computed shear forces that are developed between the slab and the stubs. This is in contrast to the current design practice for simple composite beams, which is based on the smaller of the ultimate axial load-carrying capacity of the slab and the steel beam [2, 3]. However, the simplified approach of current specifications is not applicable to members where the cross-section varies significantly along the length (nonprismatic beams). The computed shear force design approach also promotes connector economy, in the sense that a much smaller number of shear connectors is required in the interior shear transfer regions of the girder [5, 7, 21].

The stubs are welded to the top flange of the bottom chord with fillet welds. In the original uses of the system, the design called for all-around welds [11, 12]; subsequent studies demonstrated that the forces that are developed between the stubs and the bottom chord are concentrated toward the end of the stubs [5, 6, 21]. The welds should therefore be located in these regions.

The type and locations of the stub stiffeners that are indicated for the exterior stubs in Figure 18.1, as well as the lack of stiffeners for the interior stubs, represent one of the major improvements that were made to the original stub girder designs. Based on extensive research [5, 21], it was found that simple end-plate stiffeners were as efficient as the traditional fitted ones, and in many cases the stiffeners could be eliminated at no loss in strength and stiffness to the overall girder.

Figure 18.1 shows that a simple (shear) connection is used to attach the bottom chord of the stub girder to the adjacent structure (column, concrete building core, etc.). This is the most common solution, especially when a duct opening needs to be located at the exterior end of the girder. If the support is an exterior column, the slab will rest on an edge member; if it is an interior column, the slab will be continuous past the column and into the adjacent bay. This may or may not present problems in the form of slab cracking, depending on the reinforcement details that are used for the slab around the column.

The stub girder has sometimes been used as part of the lateral load-resisting system of steel-framed buildings [13, 17]. Although this has certain disadvantages insofar as column moments and the concrete slab reinforcement are concerned, the girder does provide significant lateral stiffness and ductility for the frame. As an example, the maintenance facility for Mexicana Airlines at the Mexico City International Airport, a structure utilizing stub girders in this fashion [17], survived the 1985 Mexico City earthquake with no structural damage.

Expanding on the details that are shown in Figure 18.1, Figure 18.2 illustrates the cross-section of a typical stub girder, and Figure 18.3 shows a complete girder assembly with lights, ducts, and suspended ceiling. Of particular note are the longitudinal reinforcing bars. They add flexural strength as well as ductility and stiffness to the girder, by helping the slab to extend its service range.

The longitudinal rebars are commonly placed in two layers, with the top one just below the heads of the stud shear connectors. The lower longitudinal rebars must be raised above the deck proper,

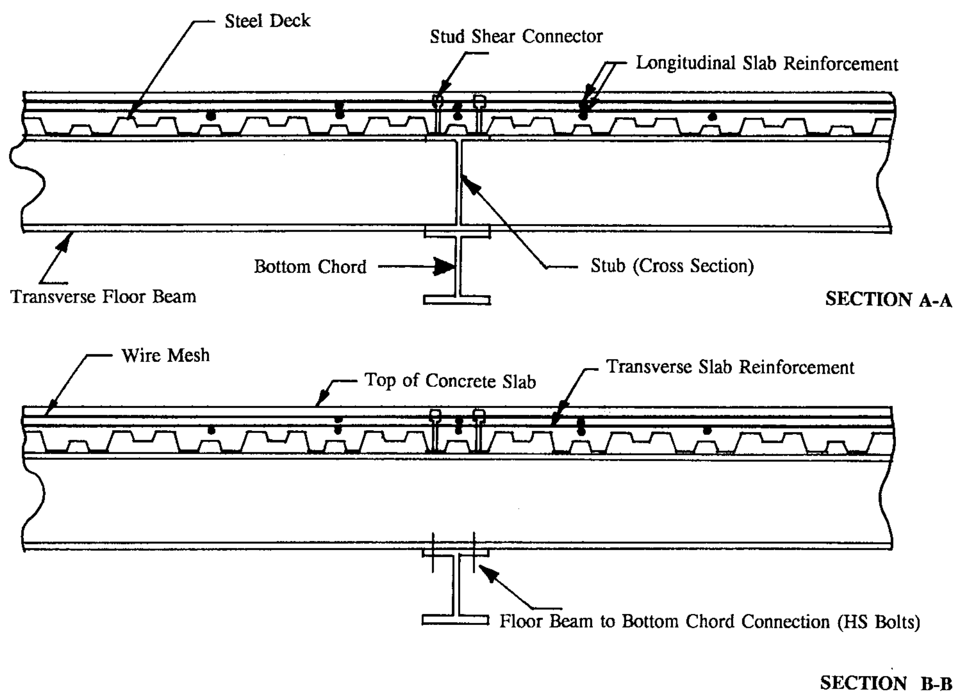

FIGURE 18.2: Cross-sections of a typical stub girder (refer to Figure 18.1 for section location).

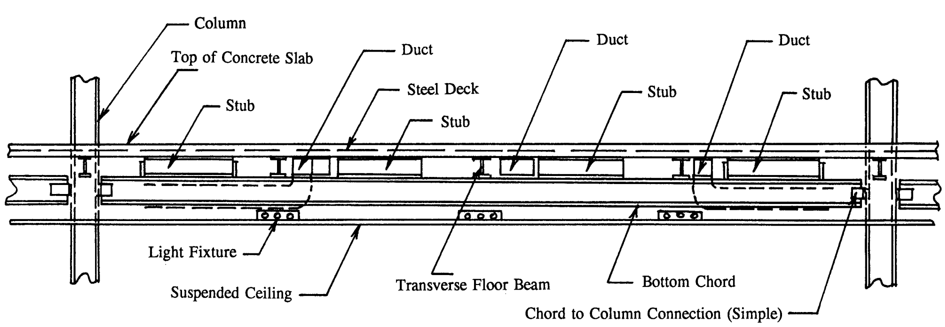

FIGURE 18.3: Elevation of a typical stub girder, complete with ductwork, lights, and suspended ceiling (duct sizes, etc., vary from system to system).

using high chairs or other means. This assures that the bars are adequately confined.

The transverse rebars are important for adding shear strength to the slab, and they also help in the shear transfer from the connectors to the slab. The transverse bars also increase the overall ductility of the stub girder, and placing the bars in a herring bone pattern leads to a small improvement in the effective width of the slab.

The common choices for stub girder floor systems have been 36- or 50-ksi-yield stress steel, with a preference for the latter, because of the smaller bottom chord size that can be used. Due to its function in the girder, there is no reason why steels such as ASTM A913 (65 ksi) cannot be used for the bottom chord. However, all detail materials (stiffeners, connection angles, etc.) are made from 36-ksi steel. Welding is usually done with 70-grade low hydrogen electrodes, using either the SMAW, FCAW, or GMAW process, and the stud shear connectors are welded in the normal fashion. All of the work is done in the fabricating shop, except for the shear connectors, which are applied in the field, where they are welded directly through the steel deck. The completed stub girders are then shipped to the construction site.

18.3 Methods of Analysis and Modeling

18.3.1 General Observations

In general, any number of methods of analysis may be used to determine the bending moments, shear forces, and axial forces throughout the components of the stub girder. However, it is essential to bear in mind that the modeling of the girder, or, in other words, how the actual girder is transformed into an idealized structural system, should reflect the relative stiffness of the elements. This means that it is important to establish realistic trial sizes of the components, through an appropriate preliminary design procedure. The subsequent modeling will then lead to stress resultants that are close to the magnitudes that can be expected in actual stub girders.

Based on this approach, the design that follows is likely to require relatively few changes, and those that are needed are often so small that they have no practical impact on the overall stiffness distribution and final member forces. The preliminary design procedure is therefore a very important step in the overall design. However, it will be shown that by using an LRFD approach, the process is simple, efficient, and accurate.

18.3.2 Preliminary Design Procedure

Using the LRFD approach for the preliminary design, it is not necessary to make any assumptions as regards the stress distribution over the depth of the girder, other than to adhere to the strength model that was developed for normal composite beams [3, 15]. The stress distribution will vary anyway along the span because of the openings.

The strength model of Hansell et al. [15] assumes that when the ultimate moment is reached, all or a portion of the slab is failing in compression, with a uniformly distributed stress of $0.85 f'_c$. The steel shape is simultaneously yielding in tension. Equilibrium is therefore maintained, and the internal stress resultants are determined using first principles. Tests have demonstrated excellent agreement with theoretical analyses that utilize this approach [5, 7, 15, 21].

The LRFD procedure uses load and resistance factors in accordance with the American Institute of Steel Construction (AISC) LRFD specification [3]. The applicable resistance factor is given by the AISC LRFD specification, Section D1, for the case of gross cross-section yielding. This is because the preliminary design is primarily needed to find the bottom chord size, and this component is primarily loaded in tension [5, 7, 10, 21]. The load factors of the LRFD specification are those of the American Society of Civil Engineers (ASCE) load standard [4], for the combination of dead plus live load.

The load computations follow the choice of the layout of the floor framing plan, whereby girder and floor beam spans are determined. This gives the tributary areas that are needed to calculate the dead and live loads. The load intensities are governed by local building code requirements or by the ASCE recommendations, in the absence of a local code.

Reduced live loads should be used wherever possible. This is especially advantageous for stub girder floor systems, since spans and tributary areas tend to be large. The ASCE load standard [4] makes use of a live load reduction factor, RF, that is significantly simpler to use, and also less conservative than that of earlier codes. The standard places some restrictions on the value of RF, to the effect that the reduced live load cannot be less than 50% of the nominal value for structural members that support only one floor. Similarly, it cannot be less than 40% of the nominal live load if two or more floors are involved.

Proceeding with the preliminary design, the stub girder and its floor beam locations determine the magnitudes of the concentrated loads that are to be applied at each of the latter locations. The following illustrative example demonstrates the steps of the solution.

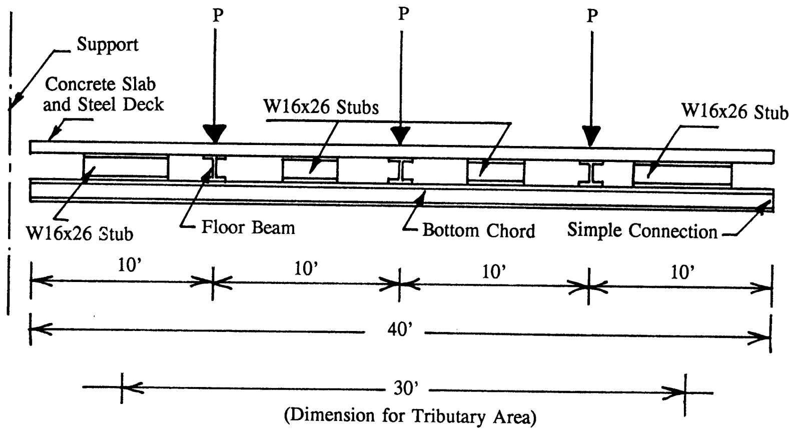

FIGURE 18.4: Stub girder layout used for preliminary design example.

EXAMPLE 18.1:

Given: Figure 18.4 shows the layout of the stub girder for which the preliminary sizes are needed. Other computations have already given the sizes of the floor beam, the slab, and the steel deck. The span of the girder is 40 ft, the distance between adjacent girders is 30 ft, and the floor beams are located at the quarter points. The steel grade remains to be chosen (36- and 50-ksi-yield stress steel are the most common); the concrete is lightweight, with $w_c = 120$ pcf and a compressive strength of $f_c' = 4000$ psi.

Solution

Loads:

Estimated dead load = 74 psf

Nominal live load = 50 psf

Live load reduction factor:

$$RF = 0.25 + 15/\sqrt{[2 \times (30 \times 30)]} = 0.60$$

Reduced live load:

$$RLL = 0.60 \times 50 = 30 \text{ psf}$$

Load factors (for D + L combination):

For dead load: 1.2

For live load: 1.6

Factored distributed loads:

$$\begin{aligned}\text{Dead Load, } DL &= 74 \times 1.2 = 88.8 \text{ psf}\\ \text{Live Load, } LL &= 30 \times 1.6 \ \ = 48.0 \text{ psf}\\ \text{Total} &= 136.8 \text{ psf}\end{aligned}$$

Concentrated factored load at each floor beam location:
Due to the locations of the floor beams and the spacing of the stub girders, the magnitude of each load, P, is:

$$P = 136.8 \times 30 \times 10 = 41.0 \text{ kips}$$

Maximum factored midspan moment:
The girder is symmetric about midspan, and the maximum moment therefore occurs at this location:

$$M_{\max} = 1.5 \times P \times 20 - P \times 10 = 820 \text{ k-ft}$$

Estimated interior moment arm for full stub girder cross-section at midspan (refer to Figure 18.2 for typical details):
The interior moment arm (i.e., the distance between the compressive stress resultant in the concrete slab and the tensile stress resultant in the bottom chord) is set equal to the distance between the slab centroid and the bottom chord (wide-flange shape) centroid. This is simplified and conservative. In the example, the distance is estimated as

$$\text{Interior moment arm: } d = 27.5 \text{ in.}$$

This is based on having a 14 series W shape for the bottom chord, W16 floor beams and stubs, a 3-in.-high steel deck, and 3-1/4 in. of lightweight concrete over the top of the steel deck ribs (this allows the deck to be used without having sprayed-on fire protective material on the underside). These are common sizes of the components of a stub girder floor system.

In general, the interior moment arm varies between 24.5 and 29.5 in., depending on the heights of the bottom chord, floor beams/stubs, steel deck, and concrete slab.

Slab and bottom chord axial forces, F (these are the compressive and tensile stress resultants):

$$F = M_{\max}/d = (820 \times 12)/27.5 = 357.9 \text{ kips}$$

Required cross-sectional area of bottom chord, A_s:
The required cross-sectional area of the bottom chord can now be found. Since the chord is loaded in tension, the ϕ value is 0.9.

It is also important to note that in the vierendeel analysis that is commonly used in the final evaluation of the stub girder, the member forces will be somewhat larger than those determined through the simplified preliminary procedure. It is therefore recommended that an allowance of some magnitude be given for the vierendeel action. This is done most easily by increasing the area, A_s, by a certain percentage. Based on experience [7, 10], an increase of one-third is suitable, and such has been done in the computations that follow.

On the basis of the data that have been developed, the required area of the bottom chord is:

$$A_s = \frac{(M_{\max}/d)}{\phi \times F_y} \times \frac{4}{3} = \frac{F}{0.9 \times F_y} \times \frac{4}{3}$$

which gives A_s values for 36-ksi and 50-ksi steel of

$$\begin{aligned} A_s &= \frac{357.9}{0.9 \times 36} \times \frac{4}{3} = 14.73 \text{ in.}^2 \;\; (F_y = 36 \text{ ksi}) \\ A_s &= \frac{357.9}{0.9 \times 50} \times \frac{4}{3} = 10.60 \text{ in.}^2 \;\; (F_y = 50 \text{ ksi}) \end{aligned}$$

Conclusions:
If 36-ksi steel is chosen for the bottom chord of the stub girder, the wide-flange shapes W12x50 and W14x53 will be suitable. If 50-ksi steel is the choice, the sections may be W12x40 or W14x38.

Obviously the final decision is up to the structural engineer. However, in view of the fact that the W12 series shapes will save approximately 2 in. in net floor system height, per story of the building, this would mean significant savings if the overall structure is 10 to 15 stories or more. The differences in stub girder strength and stiffness are not likely to play a role [7, 10, 14].

18.3.3 Choice of Stub Girder Component Sizes

Some examples have been given in the preceding for the choices of chord and floor beam sizes, deck height, and slab configuration. These were made primarily on the basis of acceptable geometries, deck size, and fire protection requirements, to mention some examples. However, construction economy is critical, and the following guidelines will assist the user. The data that are given are based on actual construction projects.

Economical span lengths for the stub girder range from 30 to 50 ft, although the preferable spans are 35 to 45 ft; 50-ft span girders are erectable, but these are close to the limit where the dead load becomes excessive, which has the effect of making the slab govern the overall design. This is usually not an economical solution. Spans shorter than 30 ft are known to have been used successfully; however, this depends on the load level and the type of structure, to mention the key considerations.

Depending on the type and configuration of steel deck that has been selected, the floor beam spacing should generally be maintained between 8 and 12 ft, although larger values have been used. The decisive factor is the ability of the deck to span the distance between the floor beams.

The performance of the stub girder is not particularly sensitive to the stub lengths that are used, as long as these are kept within reasonable limits. In this context it is important to observe that it is usually the exterior stub that controls the behavior of the stub girder. As a practical guideline, the exterior stubs are normally 5 to 7 ft long; the interior stubs are considerably shorter, normally around 3 ft, but components up to 5 ft long are known to have been used. When the stub lengths are chosen, it is necessary to bear in mind the actual purpose of the stubs and how they carry the loads on the stub girder. That is, the stubs are loaded primarily in shear, which explains why the interior stubs can be kept so much shorter than the exterior ones.

The shear connectors that are welded to the top flange of the stub, the stub web stiffeners, and the welds between the bottom flange of the stub and the top flange of the bottom chord are crucial to the function of the stub girder system. For example, the first application of stub girders utilized fitted stiffeners at the ends and sometimes at midlength of all of the stubs. Subsequent research demonstrated that the midlength stiffener did not perform any useful function, and that only the exterior stubs needed stiffeners in order to provide the requisite web stability and shear capacity [5, 21]. Regardless of the span of the girder, it was found that the interior stubs could be left unstiffened, even when they were made as short as 3 ft [7, 14].

Similar savings were realized for the welds and the shear connectors. In particular, in lieu of all-around fillet welds for the connection between the stub and the bottom chord, the studies showed that a significantly smaller amount of welding was needed, and often only in the vicinity of the stub ends. However, specific weld details must be based on appropriate analyses of the stub, considering overturning, weld capacity at the tension end of the stub, and adequate ability to transfer shear from the slab to the bottom chord.

18.3.4 Modeling of the Stub Girder

The original work of Colaco [11, 12] utilized a vierendeel modeling scheme for the stub girder to arrive at a set of stress resultants, which in turn were used to size the various components. Elastic finite element analyses were performed for some of the girders that had been tested, mostly to examine local stress distributions and the correlation between test and theory. However, the finite element solution is not a practical design tool.

Other studies have examined approaches such as nonprismatic beam analysis [6, 21] and variations of the finite element method [16]. The nonprismatic beam solution is relatively simple to apply. On the other hand, it is not as accurate as the vierendeel approach, since it tends to overlook some important local effects and overstates the service load deflections [5, 21].

On the whole, therefore, the vierendeel modeling of the stub girder has been found to give the most accurate and consistent results, and the correlation with test results is good [5, 6, 11, 14, 21]. Finally, it offers the best physical similarity with actual girders; many designers have found this to be an important advantage.

There are no "simple" methods of analysis that can be used to find the bending moments, shear forces, and axial forces in vierendeel girders. Once the preliminary sizing has been accomplished, a computer solution is required for the girder. In general, all that is required for the vierendeel evaluation is a two-dimensional plane frame program for elastic structural analysis. This gives moments, shears, and axial forces, as well as deflections, joint rotations, and other displacement characteristics. The stress resultants are used to size the girder and its elements and connections; the displacements reflect the serviceability of the stub girder.

Once the stress resultants are known, the detailed design of the stub girder can proceed. A final run-through of the girder model should then be done, using the components that were chosen, to ascertain that the performance and strength are sufficient in all respects. Under normal circumstances no alterations are necessary at this stage.

As an illustration of the vierendeel modeling of a stub girder, the girder itself is shown in Figure 18.5a and the vierendeel model in Figure 18.5b. The girder is the same as the one used for the preliminary design example. It has four stubs and is symmetrical about midspan; therefore, only half is illustrated. The boundary conditions are shown in Figure 18.5b.

The bottom chord of the model is assigned a moment of inertia equal to the major axis I value, I_x, of the wide-flange shape that was chosen in the preliminary design. However, some analysts believe that since the stub is welded to the bottom chord, a portion of its flexural stiffness should be added to that of the moment of inertia of the wide-flange shape [5, 7, 14, 21] This approach is identical to treating the bottom chord W shape as if it has a cover plate on its top flange. The area of this cover plate is the same as the area of the bottom flange of the stub. This should be done only in the areas where the stubs are placed. In the regions of the interior and exterior stubs it is therefore realistic to increase the moment of inertia of the bottom chord by the parallel-axis value of $A_f \times d_f^2$, where A_f designates the area of the bottom flange of the stub and d_f is the distance between the centroids of the flange plate and the W shape. The contribution to the overall stub girder stiffness is generally small.

The bending stiffness of the top vierendeel chord equals that of the effective width portion of the slab. This should include the contributions of the steel deck as well as the reinforcing steel bars that are located within this width. In particular, the influence of the deck is important. The effective width is determined from the criteria in the AISC LRFD specification, Section I3.1 [3]. It is noted

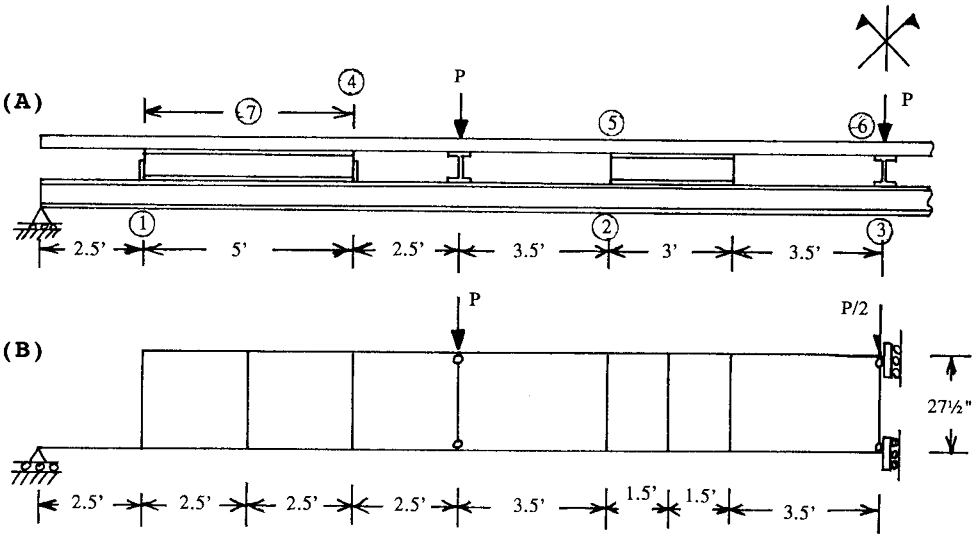

FIGURE 18.5: An actual stub girder and its vierendeel model (due to symmetry, only one half of the span is shown).

that these were originally developed on the basis of analyses and tests of prismatic composite beams. The approach has been found to give conservative results [5, 21], but should continue to be used until more accurate criteria are available.

In the computations for the slab, the cross-section is conveniently subdivided into simple geometrical shapes. The individual areas and moments of inertia are determined on the basis of the usual transformation from concrete to steel, using the modular ratio $n = E/E_c$, where E is the modulus of elasticity of the steel and E_c is that of concrete. The latter must reflect the density of the concrete that is used, and can be computed from [1]:

$$E_c = 33 \times w_c^{1.5} \times \sqrt{f_c'} \tag{18.1}$$

The shear connectors used for the stub are required to develop 100% interaction, since the design is based on the computed shear forces, rather than the axial capacity of the steel beam or the concrete slab, as is used for prismatic beams in the AISC Specifications [2, 3]. However, it is neither common nor proper to add the moment of inertia contribution of the top flange of the stub to that of the slab, contrary to what is done for the bottom chord. The reason for this is that dissimilar materials are joined, and some local concrete cracking and/or crushing can be expected to take place around the shear connectors.

The discretization of the stubs into vertical vierendeel girder components is relatively straightforward. Considering the web of the stub and any stiffeners, if applicable (for exterior stubs, most commonly, since interior stubs usually can be left unstiffened), the moment of inertia about an axis that is perpendicular to the plane of the web is calculated. As an example, Figure 18.6 shows the stub and stiffener configuration for a typical case. The stub is a 5-ft long W16x26 with 5-1/2x1/2-in. end-plate stiffeners. The computations give:

Moment of inertia about the $Z - Z$ axis:

$$\begin{aligned} I_{ZZ} &= \left[0.25 \times (60)^3\right]/12 + 2 \times 5.5 \times 0.5 \times (30)^2 \\ &= 9450 \text{ in.}^4 \end{aligned}$$

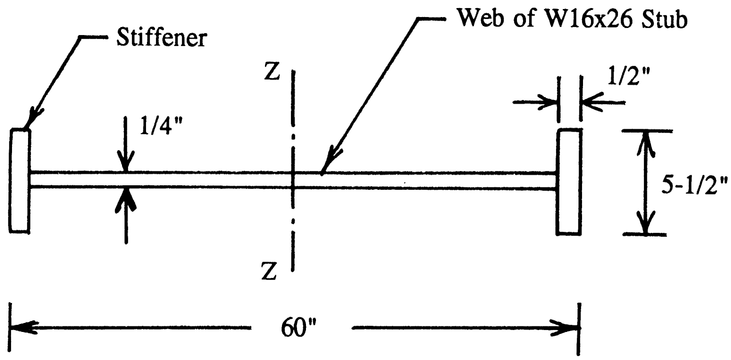

FIGURE 18.6: Horizontal cross-section of stub with stiffeners.

Depending on the number of vierendeel truss members that will represent the stub in the model, the bending stiffness of each is taken as a fraction of the value of I_{ZZ}. For the girder shown in Figure 18.5, where the stub is discretized as three vertical members, the magnitude of I_{vert} is found as:

Moment of inertia of vertical member:

$$I_{\text{vert}} = I_{ZZ}/(\text{no. of verticals}) = 9450/3 = 3150\ \text{in.}^4$$

The cross-sectional area of the stub, including the stiffeners, is similarly divided between the verticals:

Area of vertical member:

$$\begin{aligned} A_{\text{vert}} &= [A_{\text{web}} + 2 \times A_{st}]/(\text{no. of verticals}) \\ &= [0.25 \times (60 - 2 \times 0.5) + 2 \times 5.5 \times 0.5]/3 \\ &= 6.75\ \text{in.}^2 \end{aligned}$$

Several studies have aimed at finding the optimum number of vertical members to use for each stub. However, the strength and stiffness of the stub girder are only insignificantly affected by this choice, and a number between 3 and 7 is usually chosen. As a rule of thumb, it is advisable to have one vertical per foot length of stub, but this should serve only as a guideline.

The verticals are placed at uniform intervals along the length of the stub, usually with the outside members close to the stub ends. Figure 18.5 illustrates the approach. As for end conditions, these vertical members are assumed to be rigidly connected to the top and bottom chords of the vierendeel girder.

One vertical member is placed at each of the locations of the floor beams. This member is assumed to be pinned to the top and bottom chords, as shown in Figure 18.5, and its stiffness is conservatively set equal to the moment of inertia of a plate with a thickness equal to that of the web of the floor beam and a length equal to the beam depth. In the example, $t_w = 0.25$ in.; the beam depth is 15.69 in. This gives a moment of inertia of

$$\left(\left[15.69 \times 0.25^3\right]/12\right) = 0.02\ \text{in.}^4$$

and the cross-sectional area is

$$(15.69 \times 0.25) = 3.92\ \text{in.}^2$$

The vierendeel model shown in Figure 18.5b indicates that the portion of the slab that spans across the opening between the exterior end of the exterior stub and the support for the slab (a column, or a corbel of the core of the structural frame) has been neglected. This is a realistic simplification, considering the relatively low rigidity of the slab in negative bending.

Figure 18.5b also shows the support conditions that are used as input data for the computer analysis. In the example, the symmetrical layout of the girder and its loads make it necessary to analyze only one-half of the span. This cannot be done if there is any kind of asymmetry, and the entire girder must then be analyzed. For the girder that is shown, it is known that only vertical displacements can take place at midspan; horizontal displacements and end rotations are prevented at this location. At the far ends of the bottom chord only horizontal displacements are permitted, and end rotations are free to occur. The reactions that are found are used to size the support elements, including the bottom chord connections and the column.

The structural analysis results are shown in Figure 18.7, in terms of the overall bending moment, shear force, and axial force distributions of the vierendeel model given in Figure 18.5b. Figure 18.7d repeats the layout details of the stub girder, to help identify the locations of the key stress resultant magnitudes with the corresponding regions of the girder.

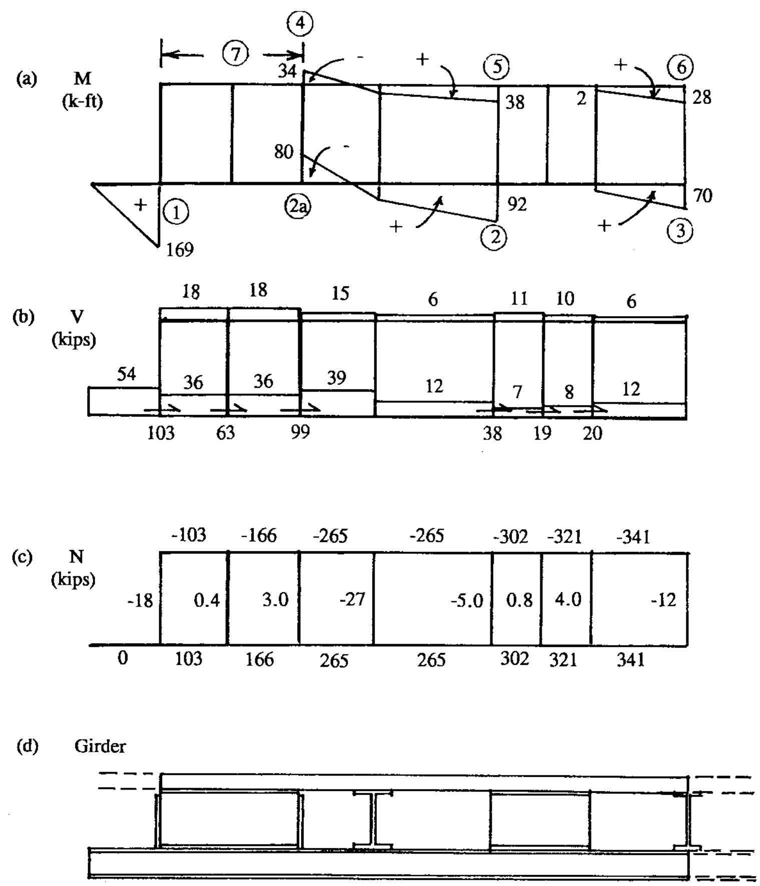

FIGURE 18.7: Distributions of bending moments, shear forces, and axial forces in a stub girder (see Figure 18.5) (dead load = 74 psf; nominal live load = 50 psf).

The design of the stub girder and its various components can now be done. This must also include deflection checks, even though research has demonstrated that the overall design will never be governed by deflection criteria [7, 14]. However, since the girder has to be built in the shored condition, the girder is often fabricated with a camber, approximately equal to the dead load deflection [7, 10].

18.4 Design Criteria For Stub Girders

18.4.1 General Observations

In general, the design of the stub girder and its components must consider overall member strength criteria as well as local checks. For most of these, the AISC Specifications [2, 3] give requirements that address the needs. Further, although LRFD and ASD are equally applicable in the design of the girder, it is recommended that LRFD be used exclusively. The more rational approach of this specification makes it the method of choice.

In several important areas there are no standardized rules that can be used in the design of the stub girder, and the designer must rely on rational engineering judgment to arrive at satisfactory solutions. This applies to the parts of the girder that have to be designed on the basis of computed forces, such as shear connectors, stiffeners, stub-to-chord welds, and slab reinforcement. The modeling and evaluation of the capacity of the central portion of the concrete slab are also subject to interpretation. However, the design recommendations that are given in the following are based on a wide variety of practical and successful applications.

It is again emphasized that the design throughout is based on the stress resultants that have been determined in the vierendeel or other analysis, rather than on idealized code criteria. However, the capacities of materials and fasteners, as well as the requirements for the stability and strength of tension and compression members, adhere strictly to the AISC Specifications. Any interpretations that have been made are always to the conservative side.

18.4.2 Governing Sections of the Stub Girder

Figures 18.5 and 18.7 show certain circled numbers at various locations throughout the span of the stub girder. These reflect the sections of the girder that are the most important, for one reason or another, and are the ones that must be examined to determine the required member size, etc. These are the governing sections of the stub girder and are itemized as follows:

1. Points 1, 2, and 3 indicate the critical sections for the bottom chord.
2. Points 4, 5, and 6 indicate the critical sections for the concrete slab.
3. Point 7, which is a region rather than a specific point, indicates the critical shear transfer region between the slab and the exterior stub.

The design checks that must be made for each of these areas are discussed in the following.

18.4.3 Design Checks for the Bottom Chord

The size of the bottom chord is almost always governed by the stress resultants at midspan, or point 3 in Figures 18.5 and 18.7. This is also why the preliminary design procedure focused almost entirely on determining the required chord cross-section at this location. As the stress resultant distributions in Figure 18.7 show, the bottom chord is subjected to combined positive bending moment and tensile force at point 3, and the design check must consider the beam-tension member behavior in this area. The design requirements are given in Section H1.1, Eqs. (H1-1a) and (H1-1b), of the AISC LRFD Specification [3].

Combined bending and tension must also be evaluated at point 2, the exterior end of the interior stub. The local bending moment in the chord is generally larger here than at midspan, but the axial force is smaller. Only a computation can confirm whether point 2 will govern in lieu of point 3. Further, although the location at the interior end of the exterior stub (point 2a) is rarely critical, the combination of negative moment and tensile force should be evaluated.

At point 1 of the bottom chord, which is located at the exterior end of the exterior stub, the axial force is equal to zero. At this location the bottom chord must therefore be checked for pure bending, as well as shear.

The preceding applies only to a girder with simple end supports. When it is part of the lateral load-resisting system, axial forces will exist in all parts of the chord. These must be resisted by the adjacent structural members.

18.4.4 Design Checks for the Concrete Slab

The top chord carries varying amounts of bending moment and axial force, as illustrated in Figure 18.7, but the most important areas are indicated as points 4 to 6. The axial forces are always compressive in the concrete slab; the bending moments are positive at points 5 and 6, but negative at point 4. As a result, this location is normally the one that governs the performance of the slab, not the least because the reinforcement in the positive moment region includes the substantial cross-sectional area of the steel deck.

The full effective width of the slab must be analyzed for combined bending and axial force at all of points 4 through 6. Either the composite beam-column criteria of the AISC LRFD specification [3] or the criteria of the reinforced concrete structures code of the American Concrete Institute (ACI) [1] may be used for this purpose.

18.4.5 Design Checks for the Shear Transfer Regions

Region 7 is the shear transfer region between the concrete slab and the exterior stub, and the combined shear and longitudinal compressive capacity of the slab in this area must be determined. The shear transfer region between the slab and the interior stub always has a smaller shear force.

Region 7 is critical, and several studies have shown that the slab in this area will fail in a combination of concrete crushing and shear [5, 6, 7, 21]. The shear failure zone usually extends from corner to corner of the steel deck, over the top of the shear connectors, as illustrated in Figure 18.8. This also

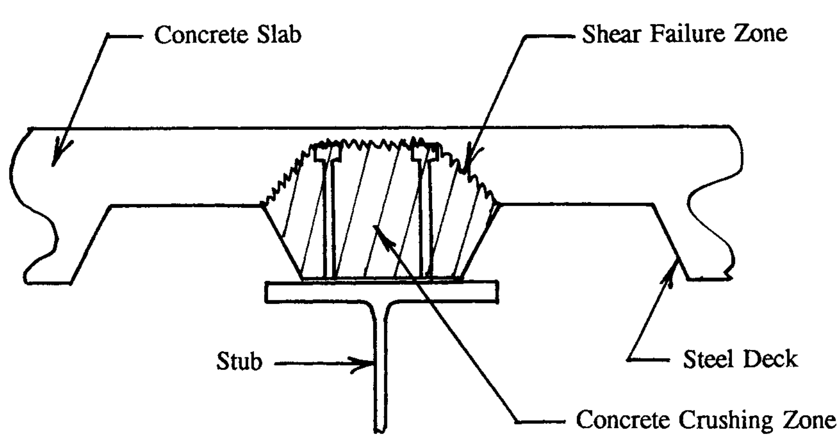

FIGURE 18.8: Shear and compression failure regions in the slab of the stub girder.

emphasizes why the placement of the longitudinal reinforcing steel bars in the central flute of the steel deck is important, as well as the location of the transverse bars: both groups should be placed just below the level of the top of the shear connectors (see Figure 18.2). The welded wire mesh reinforcement that is used as a matter of course, mostly to control shrinkage cracking in the slab, also assists in improving the strength and ductility of the slab in this region.

18.4.6 Design of Stubs for Shear and Axial Load

The shear and axial force distributions indicate the governing stress resultants for the stub members. It is important to note that since the vierendeel members are idealized from the real (i.e., continuous) stubs, bending is not a governing condition. Given the sizes and locations of the individual vertical members that make up the stubs, the design checks are easily made for axial load and shear. For example, referring to Figure 18.7, it is seen that the shear and axial forces in the exterior and interior stubs, and the axial forces in the verticals that represent the floor beams, are the following:

Exterior stub verticals:

Shear forces:	103 kips	63 kips	99 kips
Axial forces:	−18 kips	0.4 kips	3 kips

Interior stub verticals:

Shear forces:	38 kips	19 kips	20 kips
Axial forces:	−5 kips	0.8 kips	4 kips

Floor beam verticals:

$$\begin{aligned} &\text{Exterior:} \quad \text{Axial force} = -39 \text{ kips} \\ &\text{Interior:} \quad \text{Axial force} = -12 \text{ kips} \end{aligned}$$

Shear forces are zero in these members.

The areas and moments of inertia of the verticals are known from the modeling of the stub girder. Figure 18.7 also shows the shear and axial forces in the bottom and top chords, but the design for these elements has been addressed earlier in this chapter.

The design checks that are made for the stub verticals will also indicate whether there is a need for stiffeners for the stubs, since the evaluations for axial load capacity should always first be made on the assumption that there are no stiffeners. However, experience has shown that the exterior stubs always must be stiffened; the interior stubs, on the other hand, will almost always be satisfactory without stiffeners, although exceptions can occur.

The axial forces that are shown for the stub verticals in the preceding are small, but typical, and it is clear that in all probability only the exterior end of the exterior stub really requires a stiffener. This was examined in one of the stub girder research studies, where it was found that a single stiffener would suffice, although the resulting lack of structural symmetry gave rise to a tensile failure in the unstiffened area of the stub [21]. Although this occurred at a very late stage in the test, the type of failure represents an undesirable mode of behavior, and the use of single stiffeners therefore was discarded. Further, by reason of ease of fabrication and erection, stiffeners should always be provided at both stub ends.

It is essential to bear in mind that if stiffeners are required, the purpose of such elements is to add to the area and moment of inertia of the web, to resist the axial load that is applied. There is no need to provide bearing stiffeners, since the load is not transmitted in this fashion. The most economical solution is to make use of end-plate stiffeners of the kind that is shown in Figure 18.1; extensive research evaluations showed that this was the most efficient and economical choice [5, 6, 21].

The vertical stub members are designed as columns, using the criteria of Section E1 of the AISC Specification [3]. For a conservative solution, an effective length factor of 1.0 may be used. However, it is more realistic to utilize a K value of 0.8 for the verticals of the stubs, recognizing the end restraint that is provided by the connections between the chords and the stubs. The K-factor for the floor beam verticals must be 1.0, due to the pinned ends that are assumed in the modeling of these components, as well as the flexibility of the floor beam itself in the direction of potential buckling of the vertical member.

18.4.7 Design of Stud Shear Connectors

The shear forces that must be transferred between the slab and the stubs are given by the vierendeel girder shear force diagram. These are the factored shear force values which are to be resisted by the connectors. The example shown in Figure 18.7 indicates the individual shear forces for the stub verticals, as listed in the preceding section. However, in the design of the overall shear connection, the total shear force that is to be transmitted to the stub is used, and the stud connectors are then distributed uniformly along the stub. The design strength of each connector is determined in accordance with Section I5.3 of the LRFD Specification [3], including any deck profile reduction factor (Section I3.5).

Analyzing the girder whose data are given in Figure 18.7, the following is known:

Exterior stub:

Total shear force $= V_{es} = 103 + 63 + 99 = 265$ kips

Interior stub:

Total shear force $= V_{is} = 38 + 19 + 20 = 77$ kips

The nominal strength, Q_n, of the stud shear connectors is given by Eq. (I5-1) in Section I5.3 of the LRFD Specification, thus:

$$Q_n = 0.5 \times A_{sc}\sqrt{f_c' \times E_c} \leq A_{sc} \times F_u \tag{18.2}$$

where A_{sc} is the cross-sectional area of the stud shear connector, f_c' and E_c are the compressive strength and modulus of elasticity of the concrete, and F_u is the specified minimum tensile strength of the stud shear connector steel, or 60 ksi (ASTM A108).

In the equation for Q_n, the left-hand side reflects the ultimate limit state of shear yield failure of the connector; the right-hand side gives the ultimate limit state of tension fracture of the stud. Although shear almost always governs and is the desirable mode of behavior, a check has to be made to ensure that tension fracture will not take place. This as achieved by the appropriate value of E_c, setting $F_u = 60$ ksi, and solving for f_c' from Equation 18.2. The requirement that must be satisfied in order for the stud shear limit state to govern is given by Equation 18.3:

$$f_c' \leq \frac{57{,}000}{w_c} \tag{18.3}$$

This gives the limiting values for concrete strength as related to the density; data are given in Table 18.1.

For concrete with $w_c = 120$ pcf and $f_c' = 4{,}000$ psi, as used in the design example, $E_c = 2{,}629{,}000$ psi. Using 3/4-in. diameter studs, the nominal shear capacity is:

$$Q_n = 0.5\left[\pi(0.75)^2/4\right]\sqrt{(4 \times 2{,}629)} \leq \left[\pi(0.75)^2/4\right]60$$

which gives

$$Q_n = 22.7 \text{ kips } < 26.5 \text{ kips}$$

TABLE 18.1 Concrete Strength Limitations for Ductile Shear Connector Failure

Concrete density, w_c (pcf)	Maximum concrete strength, f'_c (psi)
145 ($= NW$)	4000
120	4800
110	5200
100	5700
90	6400

Note: NW = normal weight.

The LRFD Specification [3] does not give a resistance factor for shear connectors, on the premise that the ϕ value of 0.85 for the overall design of the composite member incorporates the stud strength variability. This is not satisfactory for composite members such as stub girders and composite trusses. However, a study was carried out to determine the resistance factors for the two ultimate limit states for stud shear connectors [20]. Briefly, on the basis of extensive analyses of test data from a variety of sources, and using the Q_n equation as the nominal strength expression, the values of the resistance factors that apply to the shear yield and tension fracture limit states, respectively, are:

Stud shear connector resistance factors:
Limit state of shear yielding: $\phi_{\text{conn}} = 0.90$
Limit state of tension fracture: $\phi_{\text{conn}} = 0.75$

The required number of shear connectors can now be found as follows, using the total stub shear forces, V_{es} and V_{is}, computed earlier in this section:

Exterior stub:

$$\begin{aligned} n_{es} &= V_{es}/(0.9 \times Q_n) = V_{es}/(\phi_{\text{conn}} Q_n) \\ &= 265/(0.9 \times 22.7) = 13.0 \end{aligned}$$

i.e., use $n_{es} = 14$-3/4-in. diameter stud shear connectors, placed in pairs and distributed uniformly along the length of the top flange of each of the exterior stubs.
Interior stub:

$$\begin{aligned} n_{is} &= V_{is}/(0.9 \times Q_n) = V_{is}/(\phi_{\text{conn}} Q_n) \\ &= 77/(0.9 \times 22.7) = 3.8 \end{aligned}$$

i.e., use $n_{is} = 4$-3/4-in. diameter stud shear connectors, placed singly and distributed uniformly along the length of the top flange of each of the interior stubs.

Considering the shear forces for the stub girder of Figures 18.5 and 18.7, the number of connectors for the exterior stub is approximately three times that for the interior one, as expected. Depending on span, loading, etc., there are instances when it will be difficult to fit the required number of studs on the exterior stub, since typical usage entails a double row, spaced as closely as permitted (four diameters in any direction [Section I5.6, AISC LRFD Specification [3]]). Several avenues may be followed to remedy such a problem; the easiest one is most likely to use a higher strength concrete, as long as the limit state requirements for Q_n and Table 18.1 are satisfied. This entails only minor reanalysis of the girder.

18.4.8 Design of Welds between Stub and Bottom Chord

The welds that are needed to fasten the stubs to the top flange of the bottom chord are primarily governed by the shear forces that are transferred between these components of the stub girder. The shear force distribution gives these stress resultants, which are equal to those that must be transferred between the slab and the stubs. Thus, the factored forces, V_{es} and V_{is}, that were developed in Section 18.4.7 are used to size the welds.

Axial loads also act between the stubs and the chord; these may be compressive or tensile. In Figure 18.7 it is seen that the only axial force of note occurs in the exterior vertical of the exterior stub (load = 18 kips); the other loads are very small compressive or tensile forces. Unless a significant tensile force is found in the analysis, it will be a safe simplification to ignore the presence of the axial forces insofar as the weld design is concerned.

The primary shear forces that have to be taken by the welds are developed in the outer regions of the stubs, although it is noted that in the case of Figure 18.5, the central vertical element in both stubs carries forces of some magnitude (63 and 19 kips, respectively). However, this distribution is a result of the modeling of the stubs; analyses of girders where many more verticals were used have confirmed that the major part of the shear is transferred at the ends [7, 10, 21]. The reason is that the stub is a full shear panel, where the internal moment is developed through stress resultants that act at points toward the ends, in a form of bending action. Tests have also verified this characteristic of the girder behavior [6, 21]. Finally, concentrating the welds at the stub ends will have significant economic impact [5, 7, 21].

In view of these observations, the most effective placement of the welds between the stubs and the bottom chord is to concentrate them across the ends of the stubs and along a short distance of both sides of the stub flanges. For ease of fabrication and structural symmetry, the same amount of welding should be placed at both ends, although the forces are always smaller at the interior ends of the stubs. Such U-shaped welds were used for a number of the full-size girders that were tested [5, 6, 21], with only highly localized yielding occurring in the welds. A typical detail is shown in Figure 18.9; this reflects what is recommended for use in practice.

Prior to the research that led to the change of the welded joint design, the stubs were welded with all-around fillet welds for the exterior as well as the interior elements. The improved, U-shaped detail provided for weld metal savings of approximately 75% for interior stubs and around 50% for exterior stubs.

For the sample stub girder, W16x26 shapes are used for the stubs. The total forces to be taken by the welds are:

Exterior stub: $V_{es} = 265$ kips
Interior stub: $V_{is} = 77$ kips

Using E70XX electrodes and 5/16-in. fillet welds (the fillet weld size must be smaller than the thickness of the stub flange, which is 3/8 in. for the W16x26), the total weld length for each stub is L_w, given by (refer to Figure 18.9):

$$L_w = 2(b_{fs} + 2\ell)$$

since U-shaped welds of length $(b_{fs} + 2\ell)$ are placed at each stub end. The total weld lengths required for the stub girder in question are therefore:

Exterior stub:

$$\begin{aligned}(L_w)_{es} &= V_{es}/(0.707 a \phi_w F_w) \\ &= 265/[0.707(5/16) \times 0.75(0.6 \times 70)] = 38.1 \text{ in.}\end{aligned}$$

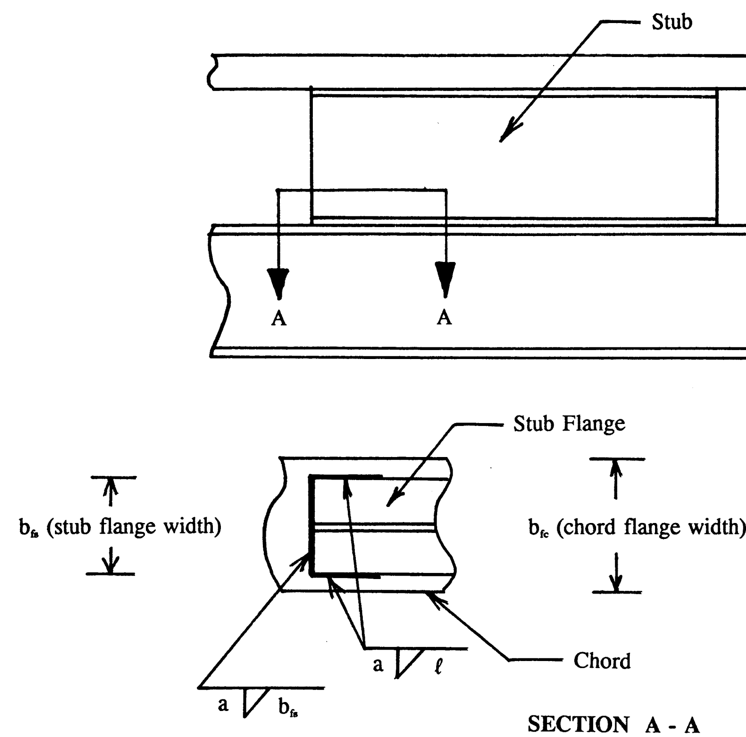

FIGURE 18.9: Placement of U-shaped fillet weld for attachment at each end of stub to bottom chord.

Interior stub:

$$\begin{aligned}(L_w)_{is} &= V_{is}/(0.707a\phi_w F_w) \\ &= 77/[0.707(5/16) \times 0.75(0.6 \times 70)] = 11.1 \text{ in.}\end{aligned}$$

In the above expressions, $a = 5/16$ in. = fillet weld size, $\phi_w = 0.75$, and $F_w = 0.6F_{EXX} = 0.6 \times 70 = 42$ ksi for E70XX electrodes (Table J2.3, AISC LRFD Specification [3]). The total U-weld lengths at each stub end are therefore:

Exterior stub: $L_{Ues} = 19.1$ in.
Interior stub: $L_{Uis} = 5.6$ in.

With a flange width for the W16x26 of 5.50 in., the above lengths can be simplified as:

$$L_{Ues} = 5.50 + 7.0 + 7.0$$

where ℓ_{es} is chosen as 7.0 in. For the interior stub:

$$L_{Uis} = 5.50 + 2.0 + 2.0$$

where ℓ_{is} is chosen as 2.0 in.

The details chosen are a matter of judgment. In the example, the interior stub for all practical purposes requires no weld other than the one across the flange, although at least a minimum weld return of 1/2 in. should be used.

18.4.9 Floor Beam Connections to Slab and Bottom Chord

In the vierendeel model, the floor beam is represented as a pinned-end compression member. It is designed using a K-factor of 1.0, and the floor beam web by itself is almost always sufficient to take the axial load. However, the floor beam must be checked for web crippling and web buckling under shoring conditions.

No shear is transferred from the beam to the slab or the bottom chord. In theory, therefore, any attachment device between the floor beam and the other components should not be needed. However, due to construction stability requirements, as well as the fact that the floor beam usually is designed for composite action normal to the girder, fasteners are needed. In practice, these are not actually designed; rather, one or two stud shear connectors are placed on the top flange of the beam, and two high-strength bolts attach the lower flange to the bottom chord.

18.4.10 Connection of Bottom Chord to Supports

In the traditional use of stub girders, the girder is supported as a simple beam, and the bottom chord end connections need to be able to transfer vertical reactions to the supports. The latter structural elements may be columns, or the girder may rest on corbels or other types of supports that are part of the concrete core of the building. For both of these cases the reactions that are to be carried to the adjacent structure are given by the analysis, and the response needs for the supports are clear.

Any shear-type beam connections may be used to connect the bottom chord to a column or a corbel or similar bracket. It is important to ascertain that the chord web shear capacity is sufficient, including block shear (Section J5 of the AISC LRFD Specification [3]).

Some designers prefer to use slotted holes for the connections, and to delay the final tightening of the bolts until after the shoring has been removed. This is done on the premise that the procedure will leave the slab essentially stress free from the construction loads, leading to less cracking in the slab during service. Other designers specify additional slab reinforcement to take care of any cracking problem. Experience has shown that both methods are suitable.

The slab may be supported on an edge beam or similar element at the exterior side of the floor system. There is no force transfer ability required of this support. In the interior of the building the slab will be continuously cast across other girders and around columns; this will almost always lead to some cracking, both in the vicinity of the columns as well as along beams and girders. With suitable placement of floor slab joints, this can be minimized, and appropriate transverse reinforcement for the slab will reduce, if not eliminate, the longitudinal cracks.

Data on the effects of various types of cracks in composite floor systems are scarce. Current opinion appears to be that the strength may not be influenced very much. In any case, the mechanics of the short- and long-term service response of composite beams is not well understood. Recent studies have developed models for the cracking mechanism and the crack propagation [18]; the correlation with a wide variety of laboratory tests is good. However, a comprehensive study of concrete cracking and its implications for structural service and strength needs to be undertaken.

18.4.11 Use of Stub Girder for Lateral Load System

The stub girder was originally conceived only as being part of the vertical load-carrying system of structural frames, and the use of simple connections, as discussed in Section 18.4.9, came from this development. However, because a deep, long-span member can be very effective as a part of the lateral load-resisting system for a structure, several attempts have been made to incorporate the stub girder into moment frames and similar systems. The projects of Colaco in Houston [13] and Martinez-Romero [17] in Mexico City were successful, although the designers noted that the cost premium could be substantial.

For the Colaco structure, his applications reduced drift, as expected, but gave much more complex beam-to-column connections and reinforcement details in the slab around the columns. Thus, the exterior stubs were moved to the far ends of the girders, and moment connections were designed for the full depth. For the Mexico City building, the added ductility was a prime factor in the survival of the structure during the 1985 earthquake.

The advantages of using the stub girders in moment frames are obvious. Some of the disadvantages have been outlined; in addition, it must be recognized that the lack of room for perimeter HVAC ducts may be undesirable. This can only be addressed by the mechanical engineering consultant. As a general rule, a designer who wishes to use stub girders as part of the lateral load-resisting system should examine all structural effects, but also incorporate nonstructural considerations such as are prompted by HVAC and electronic communication needs.

18.4.12 Deflection Checks

The service load deflections of the stub girder are needed for several purposes. First, the overall dead load deflection is used to assess the camber requirements. Due to the long spans of typical stub girders, as well as the flexibility of the framing members and the connections during construction, it is important to end up with a floor system that is as level as possible by the time the structure is ready to be occupied. Thus, the girders must be built in the shored condition, and the camber should be approximately equal to the dead load deflection.

Second, it is essential to bear in mind that each girder will be shored against a similar member at the level below the current construction floor. This member, in turn, is similarly shored, albeit against a girder whose stiffness is greater, due to the additional curing time of the concrete slab. This has a cumulative effect for the structure as a whole, and the dead load deflection computations must take this response into account.

In other words, the support for the shores is a flexible one, and deflections therefore will occur in the girder as a result of floor system movements of the structure at levels in addition to the one under consideration. Although this is not unique to the stub girder system, the span lengths and the interaction with the frame accentuate the influence on the girder design.

Depending on the structural system, it is also likely that the flexibility of the columns and the connections will add to the vertical displacements of the stub girders. The deflection calculations should incorporate these effects, preferably by utilizing realistic modified E_c values and determining displacements as they occur in the frame. Thus, the curing process for the concrete might be considered, since the strength development as a function of time is directly related to the value of E_c [1]. This is a subject that is open for study, although similar criteria have been incorporated in studies of the strength and behavior of composite frames [8, 9]. However, detailed evaluations of the influence of time-dependent stiffness still need to be made for a wide variety of floor systems and frames. The cumulative deflection effects can be significant for the construction of the building, and consequently also must enter into the contractor's planning. This subject is addressed briefly in Section 18.5.

Third, the live load deflections must be determined to assess the serviceability of the floor system under normal operating conditions. However, several studies have demonstrated that such displacements will be significantly smaller than the $L/360$ requirement that is normally associated with live load deflections [6, 7, 10, 14, 21]. It is therefore rarely possible to design a girder that meets the strength and the deflection criteria simultaneously [14]. In other words, strength governs the overall design.

Finally, although they rarely play a role in the overall response of the stub girder, the deflections and end rotations of the slab across the openings of the girder should also be checked. This is primarily done to assess the potential for local cracking, especially at the stub ends and at the floor beams. However, proper placement of the longitudinal girder reinforcement is usually sufficient to prevent problems of this kind, since the deformations tend to be small.

18.5 Influence of Method of Construction

A number of construction-related considerations have already been addressed in various sections of this chapter. The most important ones relate to the fact that the stub girders must be built in the shored condition. The placement and removal of the shores may have a significant impact on the performance of the member and the structure as a whole. In particular, too early shore removal may lead to excessive deflections in the girders at levels above the one where the shores were located. This is a direct result of the low stiffness of “green” concrete. It can also lead to “ponding” of the concrete slab, producing larger dead loads than accounted for in the original design. Finally, larger girder deflections can be translated into an “inward pulling” effect on the columns of the frame. However, this is clearly a function of the framing system.

On the other hand, the use of high early strength cement and similar products can reduce this effect significantly. Further, since the concrete usually is able to reach about 75% of the 28-day strength after 7 to 10 days, the problem is less severe than originally thought [5, 7, 10]. In any case, it is important for the structural engineer to interact with the general contractor, in order that the influence of the method of construction on the girders as well as the frame can be quantified, however simplistic the analysis procedure may be.

Due to the larger loads that can be expected for the shores, the latter must either be designed as structural members or at least be evaluated by the structural engineer. The size of the shores is also influenced by the number of floors that are to have these supports left in place. As a general rule, when stub girders are used for multi-story frames, the shores should be left in place for at least three floor levels. Some designers prefer a larger number; however, any choices of this kind should be based on computations for sizes and effects. Naturally, the more floors that are specified, the larger the shores will have to be.

18.6 Defining Terms

Composite: Steel and concrete acting in concert.

Formed steel deck: A thin sheet of steel shaped into peaks and valleys called corrugations.

Green concrete: concrete that has just been placed.

HVAC: Heating, ventilating, and air conditioning.

Lightweight: Refers to concrete with unit weights between 90 and 120 pcf.

Normal weight: Refers to concrete with unit weights of 145 lb per cubic foot (pcf).

Prismatic beam: A beam with a constant size cross-section over the full length.

Rebar: An abbreviated name for reinforcing steel bars.

Serviceability: The ability of a structure to function properly under normal operating condictions.

Shoring: Temporary support.

Vierendeel girder: A girder with top and bottom chords attached to each other through fully welded connections to vertical (generally) members.

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Further Reading

The references that accompany this chapter are all-encompassing for the literature on stub girders. Primary references that should be studied in addition to this chapter are [5, 7, 10, 11], and [13].