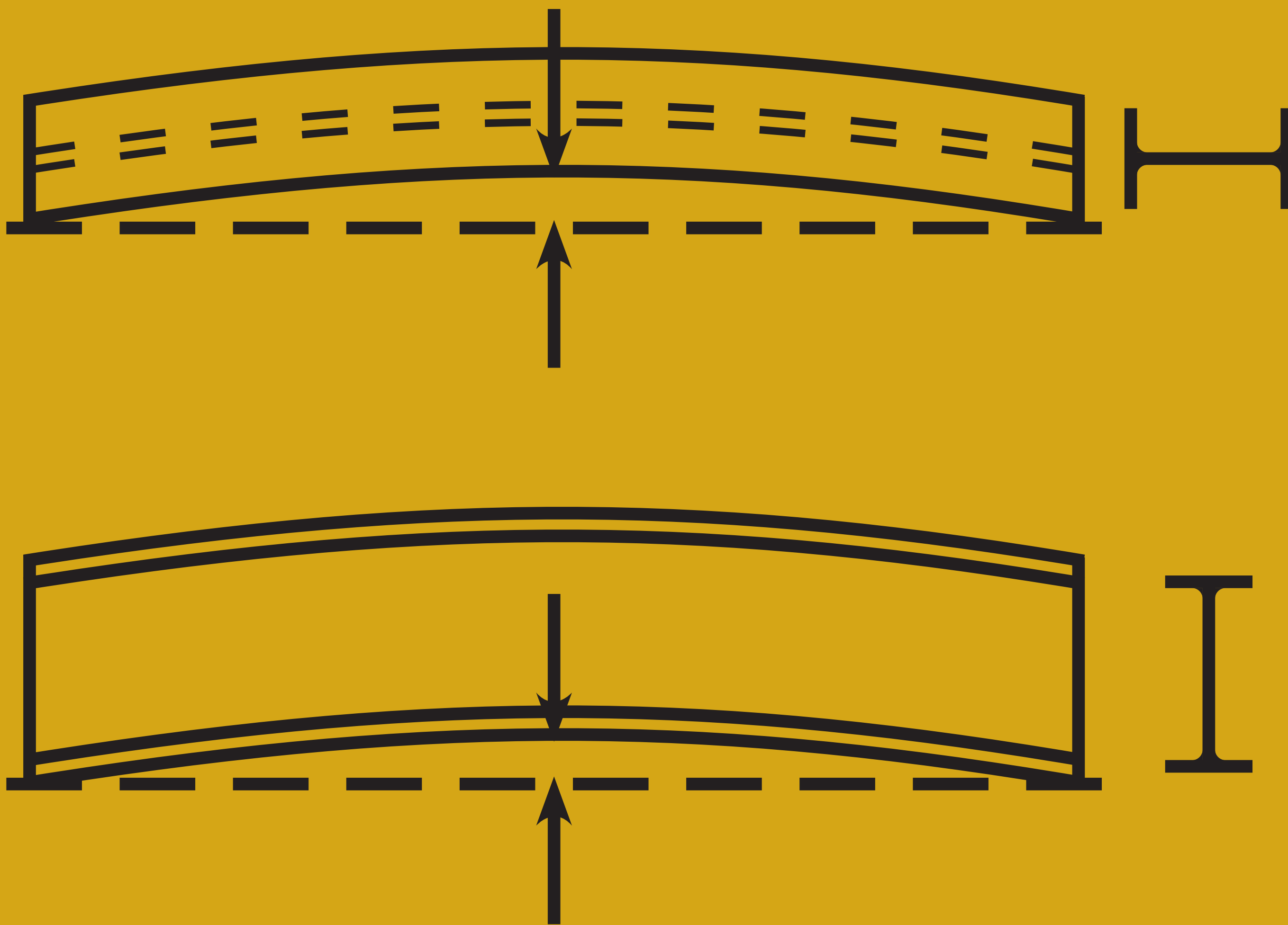

Design Guide 36

Design Considerations for Camber

Smarter.
Stronger.
Steel.

Design Guide 36

Design Considerations for Camber

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The late David Ricker, PE, was not part of this project, but he wrote the first practical paper on cambering steel beams 30 years ago. His example of sharing his knowledge in that paper and numerous others that followed set an example of professional responsibility that should inspire all of us.

Preface

This Design Guide has been developed to educate the industry on the advantages, disadvantages, and potential problems associated with specifying camber for steel floor or roof members in an effort to enable a practicing engineer to make informed decisions in evaluating the best solution with regard to camber for their specific project. Camber of members other than composite floor beams, such as transfer girders, plate girders, cantilever beams, and various types of trusses, will also be discussed.

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Chapter 1
Introduction

1.1 OBJECTIVE AND SCOPE

Advances in technology and analysis methods over the last few decades have led to more efficient structural designs and an expectation by clients that most members within the building system will be optimized. However, there is not always a single or obvious "optimal" solution. In steel building structures, it can be as much art as science, balancing the demands of strength against the demands of serviceability and economy. When designing horizontal members that support gravity loads from floors or roofs, one of the tools in the structural engineer's toolbox for dealing with these conflicting demands is incorporating camber into their designs.

Unlike many situations engineers encounter, there is no specific right or wrong answer dictating how to incorporate camber into member designs. The intention behind the development of this Design Guide is to educate the industry on some of the pros, cons, and pitfalls associated with specifying camber for steel floor or roof members to enable a practicing engineer to make informed decisions in evaluating the best solution for their specific project. There are already several published articles that address various aspects of cambering beams (Kloiber, 1989; Winters-Downey, 2006; Criste, 2009); the objective of this Design Guide, therefore, is to collate this knowledge into a single resource and expand past discussions where appropriate. By addressing the influence of camber on the design of different member types, such as beams or trusses, and explaining the fabrication and erection processes utilized to achieve the design requirements, engineers can make smarter and more cost-effective choices in optimizing building designs.

Camber design, especially for composite floor systems, is a complex steel design problem. The successful use of cambered steel beams requires the proper placement of concrete on the structure. An appendix, Floor Levelness, has been included in this Guide to provide an overview of the interaction of the steel structure with current concrete placement practices.

1.2 DEFINING CAMBER

Engineers have historically used camber to compensate for deflection due to anticipated loads. The design objective when cambering a horizontal framing member is to intentionally induce an upward curvature in the member such that when the member is subject to full loading, the member's final deflected shape stays within a desired deflection criterion. Generally, when construction is complete, the upward curvature that was introduced as camber is negated by the member deflection, and the final member geometry will be relatively flat or slightly deflected downward. However, if the designer desires a positive camber after loading, that alternative is left to the designer's judgment. The curvature associated with camber is typically not expected to be visible once all the structural components are installed in the building.

Camber is different from member curving. Member curving, also commonly referred to as member bending, entails modifying the member geometry by introducing an in-plane arch or an out-of-plane sweep for architectural appearance or function and is not addressed in this document. This specified horizontal or vertical curvature is intended to exist in the member at the completion of construction and is typically large enough in magnitude to create a visual effect. AISC Design Guide 33, *Curved Member Design* (Dowswell, 2018), provides more information and guidance for the design of curved steel members.

The term camber is also used in the provisions of ASTM A6/A6M (ASTM, 2019) when referring to the permissible out-of-straightness of steel shapes in the major or x-x axis. This tolerance is given as ⅛ in. for each 10 ft of length, except for certain sections ordered as columns that have special maximum limits. This allowable out-of-straightness can occur anywhere along the member length, and the maximum out-of-straightness may not always coincide with the member midspan. Most members are typically manufactured with less natural camber than this maximum limitation. The 2016 AISC *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2016a), hereafter referred to as the AISC *Code of Standard Practice*, states that it is important to orient this out-of-straightness (natural camber) resulting from the member manufacturing process in the positive (upward) direction for non-cambered members. For members with designed or induced camber, this slight out-of-straightness can be ignored.

For the purposes of this Design Guide, unless specifically identified as natural or mill camber, any time the authors refer to camber within the document, it will mean the specified camber imposed on the member by the design engineer through the steel fabrication process.

Camber has become more prevalent in the last couple of decades for numerous reasons. Advances in fabrication processes have made it easier and more economical for fabricators to introduce camber into members. An increase in structural steel material strength has resulted in the use of lighter and more efficient sections. Additionally, significant research in the last few decades has allowed designers greater utilization of this strength. The evolution of steel member design away from a restrictive allowable stress limitation methodology toward an approach that more explicitly recognizes the impacts of elastic and plastic member behaviors has also contributed to more efficient structural steel designs. The result is that serviceability, not strength, is just as important in specifying member sizes and, in many cases, will control the final design selection.

Members, such as composite and noncomposite beams, transfer girders, and various types of trusses and steel joists, are frequently cambered. Camber design may be based on some portion of the dead load only, dead load plus partial live load, or full dead and live load, depending on the design requirements. This Design Guide will discuss each type of member, but will focus on the design, fabrication, and quality control and quality assurance requirements for composite beams.

1.3 PURPOSE AND BENEFITS OF CAMBER

Camber is frequently associated with design of composite beams because it can provide so much benefit to those members. Composite beam construction has evolved over the last half century from a special method to achieve increased floor loadings to an essential element in many types of building construction. Designers have three basic options on how to approach the design of composite beams.

1. The beams can be sized with the stiffness required to support the weight of the wet concrete with minimum deflection to avoid ponding. Deflection limits must be carefully maintained during the precomposite construction phase and the final post-composite condition. This results in relatively large or heavy steel members.
2. The beams can be cambered to compensate for all or part of the deflection caused by placing the concrete, which results in smaller or lighter steel members.
3. The beams can be shored until the concrete achieves the strength required to act compositely with the beam and all deflections are considered using the post-composite member properties.

Cambering has always been recognized as the more economical material option over upsizing the beams and adding more concrete to compensate for deflection. However, in the 1960s and 1970s, many fabricators did not have equipment to mechanically camber beams. The process of heat cambering was in its infancy and was slow; therefore, it was expensive. The steel mills did offer mechanical cambering as an add-on option during material purchasing. Unfortunately, ordering pre-cambered beams from the mills could often be a scheduling issue. Additionally, there was some question about the accuracy of the camber based on statements in earlier editions of the AISC *Manual of Steel Construction* (AISC, 1989) about possible camber loss in shipment.

Shoring was an economical option in the early days of composite construction. However, bottom flange bracing was required, and working around the shores often created onsite schedule problems. From a serviceability perspective, another concern was that when the shores were removed, some deflection still occurred and significant cracking tended to occur over the supporting girders. During unshored construction, deflections due to the structure self-weight occurred while the concrete was still plastic and no tension stresses developed in the concrete. But in shored construction, the post-composite deflections due to the structure dead loads stress the hardened concrete when the shores were removed. Mild reinforcing steel helps to reduce the cracks and control their width, but they still tend to occur.

Large double-ram hydraulic cambering presses became commercially available in fabrication shops in the 1980s. Once fabricators were able to mechanically camber beams in-house, cambering became the obvious economical choice for most composite floor systems. There remained concerns about the best procedure to calculate the optimal camber and concerns about possible camber loss, both of which will be addressed in this Design Guide.

ASCE/SEI 7, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE, 2016), Appendix C, addresses the issue of serviceability, which includes camber. However, information in the Appendix is considered cautionary, not mandatory, so there is no obligation on the part of designers to follow it explicitly. Appendix Section C.4, Camber, states: "Special camber requirements that are necessary to bring a loaded member into proper relationship with the work of other trades shall be set forth in the design documents. Beams detailed without specified camber shall be positioned during erection so that any minor camber is upward. If camber involves erection of any member under preload, this shall be noted in the design documents." The associated Commentary Section CC.4, Camber, states: "When required, camber should be built into horizontal structural members to give proper appearance and drainage and to counteract anticipated deflection from loading and potential ponding."

The Commentary in ASCE/SEI 7, Section CC.2.1, Vertical Deflections, discusses appropriate limiting values of deformation and notes that they depend on the type of structure, detailing, and intended use. The deformations and limits discussed are based on service loads applied to the completed structure. Deformations such as those that occur in composite beams during construction are not specifically covered, but if they exceed specified cambers, that variance should be added to the service load deformations when evaluating the serviceability of the structure.

As designers have become more comfortable designing and specifying members utilizing camber as a tool to offset anticipated deflections, it is natural that they would explore options of applying this concept to other types of structural members as well. Camber of members other than composite floor beams, such as transfer girders, plate girders, cantilever beams, and various types of trusses, will also be discussed in this Design Guide.

Chapter 2
Mechanical Principles

2.1 ELASTIC-PLASTIC STRAINS

Cambering beams is a simple process that requires local plasticization of a relatively small length of the steel member to achieve the required shape. This is accomplished mechanically by force using special presses or by use of local heats applied to sections of the member to upset and shorten part of the section. Mechanical cambering is by far the most economical and common method of cambering beams, and the principles are the same as those used to curve members, as discussed in AISC Design Guide 33, *Curved Member Design* (Dowswell, 2018). The magnitude of the inelastic strains required to achieve the permanent deformation associated with camber, however, is significantly less than for curving a member because the radius of curvature required for camber is substantially larger than that required for most architectural bending.

When mechanically cambering, the elastic-inelastic behavior of the steel, as shown in Figure 2-1, must be considered. If steel is strained into the plastic or yielding range but less than the strain hardening range, the mechanical properties after unloading will be approximately the same as the virgin material (Brockenbrough and Merritt, 2006). The relatively large radius of curvatures for typical design cambers requires inelastic strains that are significantly less than the strain hardening level.

When calculating the required strain to camber a beam, the typical elastic moment curvature equations do not apply in the inelastic region, and it is necessary to go to the strain curvature relationship shown in Figure 2-2 (Bjorhovde, 2006). This assumes that plane sections remain plane, which is sufficiently accurate for the behavior of cambered members.

For doubly symmetric shapes for a given radius of curvature, R, the maximum strain, ε_{max}, in the cross section is:

$$\varepsilon_{max} = \frac{d/2}{R} = \frac{d}{2R} \qquad (2\text{-}1)$$

and the yield strain, ε_y, is:

$$\varepsilon_y = \frac{F_y}{E} \qquad (2\text{-}2)$$

where

E = modulus of elasticity, ksi

R = radius of curvature, in.

d = depth of the section, in.

The maximum strain can also be expressed as a multiple of yield strain (Bjorhovde, 2006):

$$\varepsilon_{max} = \alpha\varepsilon_y \qquad (2\text{-}3)$$

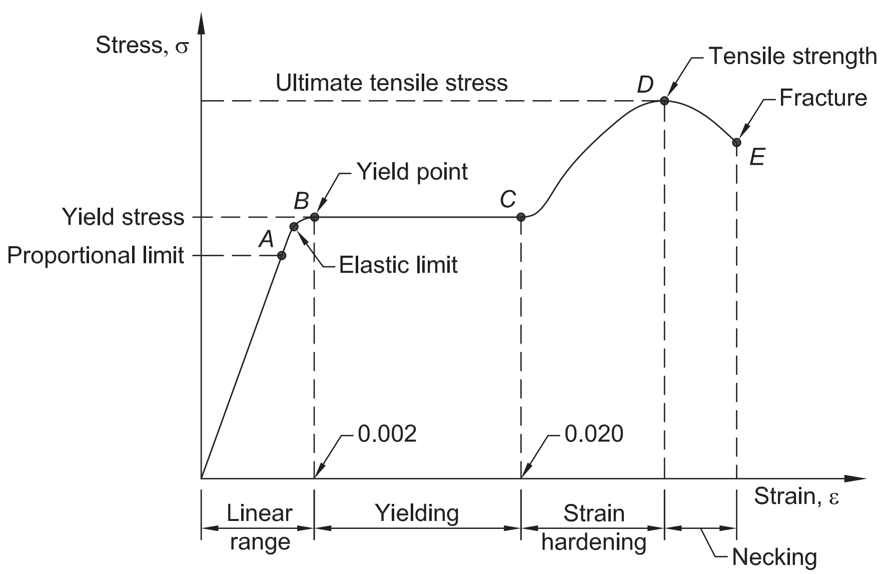


Fig. 2-1. Stress-strain curve for typical mild structural steel.

where the maximum strain factor, α, is:

$$\alpha = \left(\frac{d}{2R}\right)\left(\frac{E}{F_y}\right) \tag{2-4}$$

Strain hardening for ASTM A992/A992M material begins at approximately α = 10, as shown in Figure 2-3. When cambering beams, typically all of the curving occurs in the center of the beam over a beam segment approximately ¼ to ⅓ of the overall member length, while the outside tangent segments remain straight. This is a result of both the machine setup and the most efficient method of achieving the design camber. Depending on the member length and the magnitude of camber, the fabricator may impose multiple iterations of low-level strain at intervals within this center portion of the member to achieve the desired curvature outcome. This results in a member geometry that approximates a parabola like the theoretical deflection curve. The range of strains induced in the steel material during cambering is well below the range where strain hardening occurs. The material properties therefore are approximately the same as the original specified material.

Figure 2-4 is an example of the typical geometry for a cambered beam. A simple approximate solution for the radius of the required curve is possible by noting that for very small angles, the sine and tangent are approximately equal, and their values are approximately equal to the size of the angle, θ. The dimensions shown in Figure 2-4 are as follows:

$$R = \frac{c/2}{\sin(\theta/2)} \tag{2-5}$$

The vertical rise, y, for the straight tangent leg is:

$$y = \left(\frac{3}{8}L\right)\left[\tan(\theta/2)\right] \tag{2-6}$$

Because $\tan(\theta/4) \cong \left(\frac{1}{2}\right)\left[\tan(\theta/2)\right]$, the vertical offset within the curved beam segment, b, is:

$$b = \left(\frac{c}{2}\right)\left[\tan\left(\frac{\theta}{4}\right)\right] = \left(\frac{c}{2}\right)\left(\frac{1}{2}\right)\left[\tan\left(\frac{\theta}{2}\right)\right] \tag{2-7}$$

The total camber, Δ_c, is equal to the sum of $y + b$:

$$\Delta_c = y + b = \left(\frac{3}{8}L\right)\left[\tan\left(\frac{\theta}{2}\right)\right] + \left(\frac{c}{2}\right)\left(\frac{1}{2}\right)\left[\tan\left(\frac{\theta}{2}\right)\right] \tag{2-8}$$

Example 2.1.1—Geometric Evaluation of Camber Radius of Curvature

Given:

Determine the specific radius of curvature for a W21×44 beam spanning 40 ft with a specified camber of 2 in. The beam is ASTM A992 material. The beam geometry is illustrated in Figure 2-4.

Solution:

From AISC *Steel Construction Manual* (AISC, 2017), hereafter referred to as the AISC *Manual*, Table 2-4, the material properties are as follows:

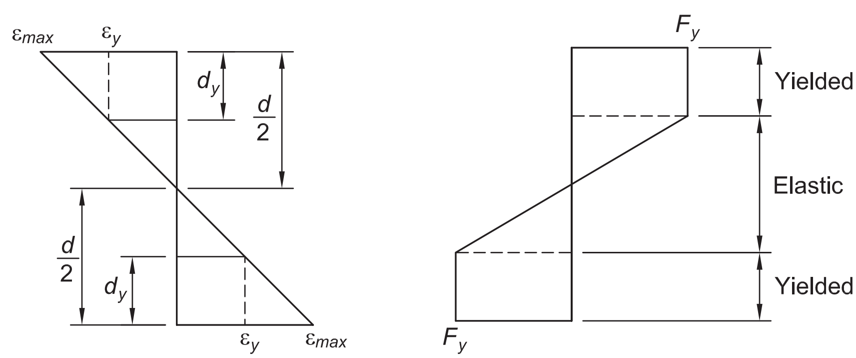


Fig. 2-2. Typical stress and strain diagrams for steel members.

ASTM A992/A992M
$F_y = 50$ ksi
$F_u = 65$ ksi

Use the given beam camber, Δ_c, and span length, L, to solve for the unknown variables.

$\Delta_c = 2$ in.

$$
\begin{aligned}
L &= (40 \text{ ft})(12 \text{ in./ft}) \\
&= 480 \text{ in.}
\end{aligned}
$$

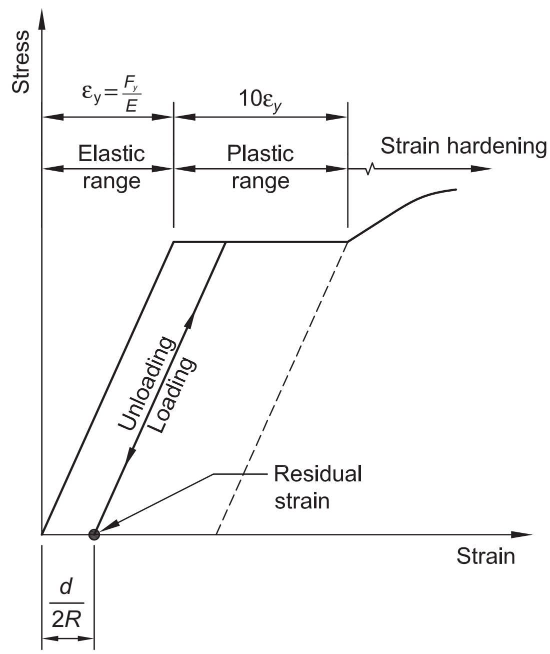


Fig. 2-3. Idealized stress-strain curve—showing loading and unloading.

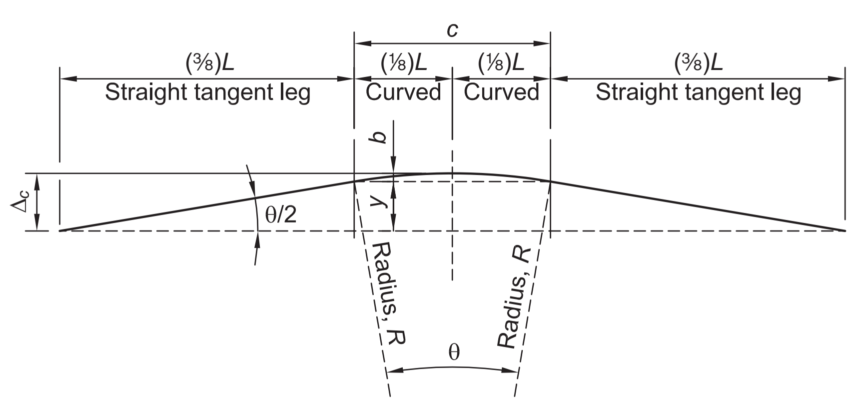


Fig. 2-4. Example of cambered beam geometry.

From Figure 2-4:

$$\begin{aligned} c &= \frac{1}{4}L \\ &= \frac{1}{4}(480\text{ in.}) \\ &= 120\text{ in.} \end{aligned}$$

$$\Delta_c = \left(\frac{3}{8}L\right)\left[\tan\left(\frac{\theta}{2}\right)\right] + \left(\frac{c}{2}\right)\left(\frac{1}{2}\right)\left[\tan\left(\frac{\theta}{2}\right)\right] \tag{2-8}$$

$$2\text{ in.} = \left(\frac{3}{8}\right)(480\text{ in.})\left[\tan\left(\frac{\theta}{2}\right)\right] + \left(\frac{120\text{ in.}}{2}\right)\left(\frac{1}{2}\right)\left[\tan\left(\frac{\theta}{2}\right)\right]$$

Solving for $\theta/2$:

$$\theta/2 = 0.546°$$

And the radius of curvature, R, is:

$$R = \frac{c/2}{\sin\left(\frac{\theta}{2}\right)} \tag{2-5}$$

$$\begin{aligned} &= \frac{120\text{ in.}/2}{\sin 0.546°} \\ &= (6{,}300\text{ in.})(1\text{ ft}/12\text{ in.}) \\ &= 525\text{ ft} \end{aligned}$$

Example 2.1.2—Determine the Maximum Strain Factor for the Beam in Example 2.1.1

Given:

Determine the maximum strain factor for the cambered beam given in Example 2.1.1.

$E = 29{,}000$ ksi

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992/A992M
$F_y = 50$ ksi
$F_u = 65$ ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

W21×44
$d = 20.7$ in.

From Example 2.1.1:

$R = 6{,}300$ in.

The maximum strain factor, α, is calculated using Equation 2-4.

$$\alpha = \left(\frac{d}{2R}\right)\left(\frac{E}{F_y}\right) \tag{2–4}$$

$$= \left[\frac{20.7 \text{ in.}}{2(6{,}300 \text{ in.})}\right]\left(\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}\right)$$

$$= 0.953$$

The radius of curvature for a beam spanning 40 ft with a 2-in. camber results in a maximum strain factor, α, close to 1. This confirms that the residual strain required to achieve the specified camber is substantially less than the strain at $\alpha = 10$, which is the value associated with strain hardening. When the bending force is removed, the material will recover elastically along a slope parallel to the original stress-strain curve and have material properties similar to those prior to cambering as shown in Figure 2-3.

The residual strain calculation shown here assumes a uniform circular curve. The double-press cambering machine shown in Figure 3-2 applied at multiple locations will produce a similar curve. A single-press machine, as shown in Figure 3-4, produces a segmented curve with larger strains localized at the load points. These strains are still well below the strain hardening region (Gergess and Sen, 2007) and can be reduced by increasing the number of segments.

Residual strains will vary depending on the camber required, the method of cambering, and the depth of section. But for typical cambers and beam sizes, the strains will always be well below the strain hardening point.

2.2 RESIDUAL STRESSES

The 9th Edition of the AISC *Manual of Steel Construction* (AISC, 1989) contained guidance on standard mill practice for cambering of rolled steel beams. Recommended maximum and minimum cambers for various beam depths and spans were given for beams cold cambered by gagging at the mill as shown in Figure 2-5. Also included was the following statement: "Camber is measured at the mill and will not necessarily be present in the same amount in the section of beam as received due to release of stress induced during the cambering operation. In general, 75% of the specified camber is likely to remain." When the 1992 AISC *Code of Standard Practice* (AISC, 1992) added provisions for camber tolerances in Section 6.4.5, it included a similar provision stating, "Members received from the rolling mill with 75% of the specified camber require no further cambering."

The rationale for this loss of camber is the release of the residual stresses induced in cambering. There was no research or data provided to verify this. Residual stresses are brought into the members because of manufacturing and fabrication operations; they are self-equilibrating stresses that are thermally and/or mechanically induced into the member. Thermally induced stresses in rolled shapes are caused by uneven cooling of the material after hot rolling. These stresses can be modified mechanically by roller straightening and/or gagging the section at the mill. Gagging is similar to cambering where a shape is loaded to yield by a large hydraulic ram centered between two supports. They can be further modified by fabrication operations such as welding and cold bending.

Figure 2-6 shows idealized residual stresses due to the mill rolling processes. The actual stresses will vary from member to member depending on the specific rolling process and the rate of cooling, which will vary based on material thickness. The forces equilibrate across the flanges with tension (+) in the center and compression (−) at the edges.

There has been limited study of residual stresses resulting from cold cambering of wide-flange sections. The

Fig. 2-5. Wide-flange section in gagging press.

residual stresses that result from cold bending will modify the residual stresses that exist due to hot rolling at the mill. Figure 2-7 illustrates the expected modified residual stress due to combining the hot-rolled residual stress of $0.35F_y$ with the residual stress of $0.35F_y$ from cold bending for the curvature range shown (Spoorenberg et al., 2011). The beam in the previous camber example has an $R/d \approx 300$ and would require less strain; however, Spoorenberg et al. indicates that with an increase in the bending ratio, R/d, the reduction in residual stresses will probably be slight.

The residual stresses are self-equilibrating stresses that do not change the ultimate strength of the beam, and except for a slight effect on the buckling strength of compression members, they are not considered in member design. AISC Design Guide 33 contains further discussion of residual stress.

2.3 CAMBER LOSS

Possible camber loss caused by residual stress modification due to handling and shipping has been a concern over the years, and the 2016 AISC *Code of Standard Practice* has commentary about the release of stresses in members over time.

If we conservatively assume that the peak residual tensile stress after cambering is approximately $0.7F_y$, as shown in Figure 2-7, then the camber in Example 2.1.1 would require a concentrated load at the center of approximately 10 kips to reach yield at the theoretical peak stress location, while most of the flange would remain elastic. Additional load would be required to strain the tension flange into yield and change the camber.

Vibration of the members in shipping has been mentioned in the past as a possible reason for loss of camber. The loss of camber due to vibration should not occur with cambered beams based on a study of effectiveness of vibratory stress relief (Dawson and Moffat, 1980). The report states that the vibratory stress amplitude needs to be more than the 0.2% proof stress to be effective. Below this strain level, no residual stress relief can be achieved. The member must be loaded to strain into the plastic region and free to deform to change residual stresses.

A limited study of the possible loss in camber in beams cambered at the mill before being received by the fabricator was done by Larson and Huzzard (2003). A total of 18 beams, W24×55 and W16×31, 30 ft long, were cambered ⅝ in. and ⅞ in., respectively, at the mill, then shipped to a fabricator. The differences in camber measured in the mill and after being unloaded at the fabricator varied from 0 to −⅛ in. Measurements were taken using a string line and foot rule to the nearest 1⁄16 in. Considering the accuracy of the measurement procedure using a string line, it is reasonable to assume there was no appreciable loss in camber. What is more important is that in this study, the members received by the fabricator had cambers that averaged more than 3⁄16 in. larger than the specified design camber.

Normal handling, blocking, and shipping by trucks of typical cambered beams to the project site should not impose the magnitude of load, either mechanical or vibratory, required to plastically strain the member and cause a loss of camber. Cambers measured in the shop can be expected to remain when received on site. Members that are slender may, however, require special handling and blocking to prevent damage in shipment.

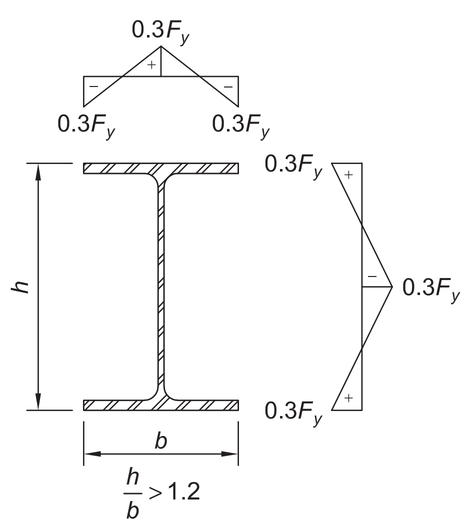


Fig. 2-6. Residual stresses due to hot rolling a wide flange.

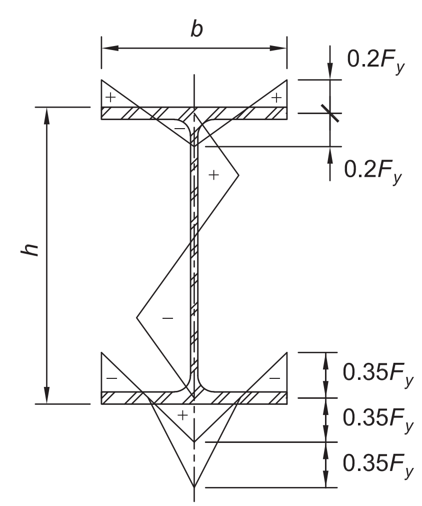


Fig. 2-7. Residual stresses due to cold bending ($10 \le R/d \le 40$) (Spoorenberg et al., 2011).

Chapter 3
Types of Camber

3.1 ASTM A6/A6M BEAM TOLERANCE (NATURAL OR MILL CAMBER)

When beams are formed at a steel mill, steel material is first cast into a slab with thickened edges that resembles a dog bone. These near-shape castings are heated and run through a rolling press that flattens and molds the malleable steel material into the desired cross-sectional shape. After the rolling process is complete and the material has cooled, the steel shape is straightened to ensure the member curvature does not exceed the out-of-straightness tolerances required by ASTM A6/A6M (ASTM, 2019). ASTM A6/A6M permits an out-of-straightness for wide-flange beams of ⅛ in. times the member length in feet divided by 10, and this allowable out-of-straightness is referred to as natural or mill camber.

$$\Delta_{max} = \tfrac{1}{8}\left(\text{number of feet of total length}/10\right) \qquad (3\text{-}1)$$

There are a few things to note when discussing natural camber. The greatest deviation from a theoretically straight beam centerline may not coincide precisely with the midspan of the member but could occur anywhere along the member length. The beam geometry may not be as simple as shown in Figure 3-1(a), but could have multiple slopes along the member length, as shown in Figure 3-1(b). This is acceptable if it does not exceed the tolerance envelope. While it is possible for a beam to have a natural camber present with an out-of-straightness on only one side of the beam centerline, it is also possible to have a beam with out-of-straightness altering from side-to-side along the member length as shown in Figure 3-1(c).

When a steel beam arrives from the mill at the steel fabricator's shop and no beam camber is specified for the member, the fabricator will examine the beam and decide which direction has the predominant out-of-straightness and designate the top flange of the beam to correspond to that direction. When the beam is oriented with the web vertical, any out-of-straightness should be an upward, or positive, deviation from a straight line. Provided the magnitude of the out-of-straightness is within the tolerance limits established by ASTM A6/A6M, these natural cambers, both positive and negative, are acceptable and can be neglected in the beam design and fabrication processes.

When a designer imposes a specific initial camber into the member to counterbalance some portion of the anticipated structural deflections, the desired camber must be indicated on the contract documents in accordance with the AISC *Code of Standard Practice*. Any natural camber that exists in the member upon delivery to the fabricator from the mill is not a concern when cambering because those initial out-of-straightness variations will be superseded by the imposed camber. To achieve the specified camber, the fabricators will utilize either a cold (mechanical) process or a heat-inducing process, both of which will be discussed in detail in the following sections.

The AISC *Code of Standard Practice* Section 6 identifies permissible tolerances applicable to induced cambers of −0 in. to +½ in. for beams equal to or less than 50 ft, with another +⅛ in. allowed for each additional 10 ft in length for beams over 50 ft. Designers should be cognizant of these tolerances when specifying cambers, especially when selecting slab thickness and finishing requirements for concrete floor systems.

3.2 COLD (MECHANICAL) CAMBER

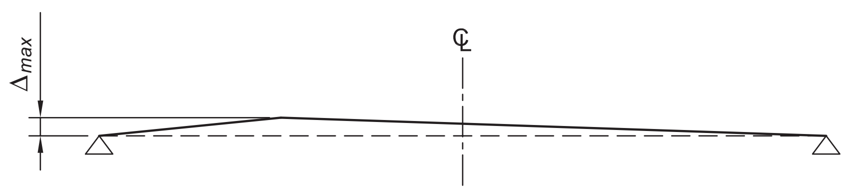
The 1949 AISC *Specification* (AISC, 1949) Section 30(c) stated "Specified Camber for rolled beams over 15 inches in depth shall be only that offered as cold cambered at the mill." The 5th Edition of the AISC *Manual* (AISC, 1946), however, indicated that mills limited camber to wide-flange sections 21-in. deep and greater and prohibited reverse or compound cambers. The mills would utilize gagging presses, such as the one shown in Figure 2-5, to impose discrete point loads to achieve the camber. The introduction of composite design in the 1961 AISC *Specification* (AISC, 1961) made cambering much more common and increased the need for fabricators to camber more efficiently. Due to demand, the mills did adjust their limits to accommodate all of the composite beam sizes, but the process was still inefficient.

The first cambering presses in fabricating and bender shops were similar in concept to the gagging presses used by the mills to camber. They consisted of a heavy steel frame with a large hydraulic ram positioned midway between two reaction points. The beam was positioned horizontally and loaded by the ram so that a plastic hinge formed at the load point. Depending on the travel of the ram, the beam was strained into the plastic region until the required permanent set was achieved. The plastically strained area was localized at the load point, which required imposing larger strains than would have been required for a continuously curved member to achieve the required camber. Typically, it was necessary to move the beam and use multiple load points to achieve the required camber curve while keeping strain levels well below strain hardening. The size of the ram required to yield the member often limited the size of the beam that could be cambered.

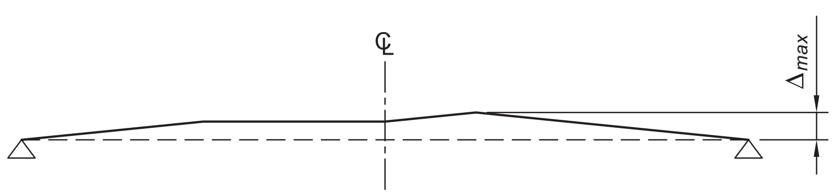
Bob Matlock, a fabrication engineer, in Houston, Texas, patented a double-ram cambering machine that revolutionized mechanical cambering in the fabrication shop in 1984. The Cambco machine shown in Figure 3-2 used a similar frame to the earlier camber presses, but had two rams located at approximately the ⅓ points between the reaction points. This had the advantage of increasing the bending capacity of the machinery and increasing the length of the beam's strained area to the distance between the two rams. This resulted in a smoother curve and a reduction in the required strain.

Since the introduction of the Cambco type machine shown in Figure 3-2, other manufacturers have developed similar pieces of cambering machinery utilizing the double-ram concept. Figure 3-3 is a generic illustration of a typical double-ram cambering machine. The rams that apply the load are spaced 6 to 8 ft apart, and the frame reaction points are typically 20 to 24 ft apart. To ensure the beam is securely anchored in the equipment frame, some nominal overhang beyond the center of the reaction point is required. Because of the physical parameters of most typical cambering machines, short beams cannot be cambered using the cold bending, or mechanical, method. Therefore, it is recommended that designers do not specify beam cambers for members that are less than 25 ft long without determining the limitations of the equipment used by their local fabricators. There are single-press machines as shown in Figure 3-4 that have the ability to camber varying length members. If it is necessary to camber a short member that cannot fit in the available cold-bending machinery, heat cambering, which will be discussed further in the following section, may be an alternative. However, heat cambering is costlier, and in many cases, it would be more economical to increase the beam size as opposed to specifying heat camber for a beam spanning a short distance. A rule of thumb for the heat cambering cost of a 20-ft-long beam would be the equivalent of adding 10 lb/ft in beam weight.

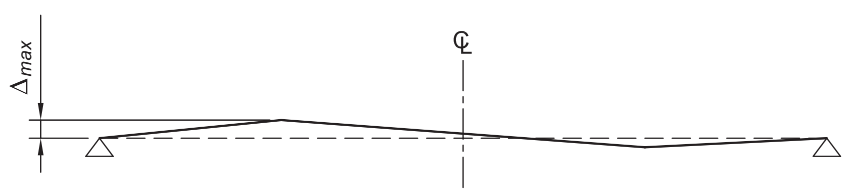
Both single-ram and double-ram presses include restraining devices at load points to control lateral-torsional buckling, and care must be taken with the member to prevent web crippling at the load points due to the high shear loads combined with bending. The bending loads required for the double-ram system are lower, and therefore web crippling is less likely. Crippling can still be a concern for very thin webs, especially for machines with large capacity rams. It is recommended that beams with webs less than ¼ in. thick not be cambered or be cambered using heat. It is typically more cost effective to use a larger beam than requiring camber in these thin-web members. A detailed comparison of the two methods, including a procedure for calculating required



(a) Maximum out-of-straightness off-center



(b) Out-of-straightness offset to one side of beam axis



(c) Out-of-straightness offset to both sides of beam axis—reverse curvature

Fig. 3-1. Acceptable natural camber profiles.

Fig. 3-2. Cambco cambering machine with conveyor (Hydradyne, LLC – Parker Fluid Products).

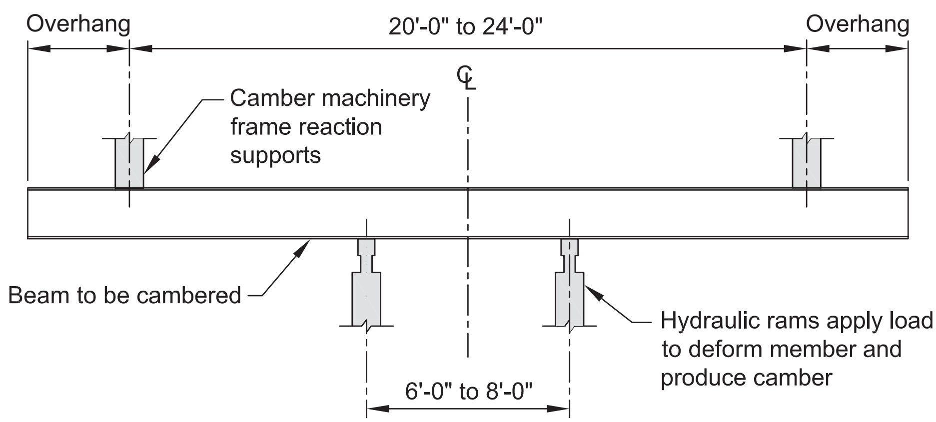


Fig. 3-3. Generic cambering machinery proportions.

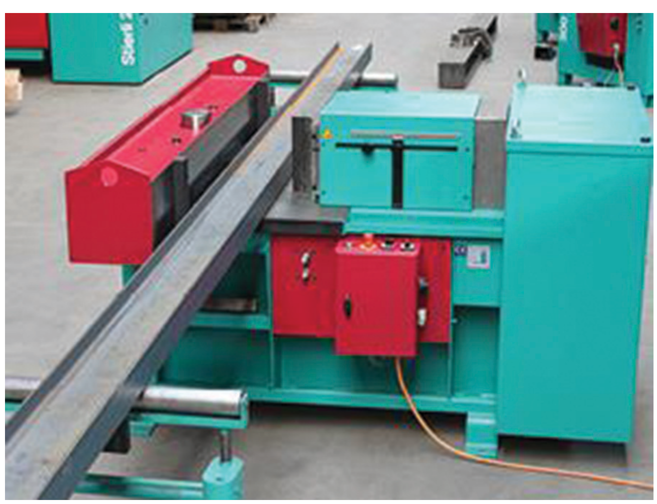


Fig. 3-4. Single-press cambering machine (Stierli-Bieger AG).

loads and strains along with a numerical example, can be found in a paper by Gergess and Sen (2007).

It is important that any holes that are required in the flange in the areas that are plastically strained in tension should be made after cambering to prevent a fracture of the net section, examples of which can be seen in Figure 3-5 (Bjorhovde, 2008). When digitally modeling members, the hole-making instructions are typically downloaded directly to the shop equipment, meaning that these holes will automatically be made before cambering. When this occurs, the holes should be carefully inspected for any sign of fracture. If possible, the design should avoid holes in the center third of the top flange length for beams to be cambered.

Mechanical/cold cambering in the fabrication shop is typically a trial-and-error process where the machine operator uses judgment to advance the ram or rams a trial distance and then releases the pressure and checks the permanent set. The machine operator will then either reapply the load or move the piece to a new load application. Most cambering machines are now equipped with roller conveyors to facilitate loading, unloading, and positioning the load point. Some newer models have incorporated measuring devices to check the camber, but typically this is still done with a string line and tape. The accuracy of the measurement is approximately ±⅛ in. The measurement should be recorded to show compliance with the specified tolerances.

The paper by Ricker (1989) cautioned against immediately reversing the member in the machine to reduce the camber in a case where the member exceeded the ±½-in. tolerance allowed. The authors of this Guide can find no basis for this restriction. If strains are on the lower portion of the yield plateau, reversal to reduce camber should not change the material properties nor produce work hardening. There is also the option of using heat to reduce the camber, as will be discussed further in the following section.

Observations in the shop indicate that for a double-press Cambco-style machine, the top or tension flange will strain slightly more than the bottom or compression flange. This could be due to the longitudinal restraint between the fixed position of the two rams that are bearing against the member. Most beams have connection holes (except critical flange holes as discussed previously) made before cambering to accommodate the shop computer numerical control (CNC) material handling and hole-making equipment. The resulting slope of the holes shown in Figure 3-6 is due to camber and differential straining and can be a fit-up concern. For long-span members with cambers of approximately 3 in., this can amount to a difference of ½ in. or more over the depth of the connection. Experience has shown that this end slope can be accommodated in erection by use of short-slotted holes or flexible-angle connections. This end slope is the reason that it is not recommended to camber beams that require vertical ends for moment, torsional, or brace connections that require connecting to the flanges.

While machines are now made that are capable of cambering heavy steel beams, such as a W36×300, machines with capacities to camber beams up to approximately a W24×84 are more typically found in fabrication shops. Where cambers are required for members larger than the available machine capacity of a shop, fabricators may have to ship the beam to another shop for cambering, which adds cost, or in special cases, heat cambering can be used.

3.3 HEAT-INDUCED CAMBER

3.3.1 Physical Principles

The physical factors involved in heat cambering have been known for some time, but because the successful application depends on the know-how and technique of the craftsperson,

Fig. 3-5. Net section fractures due to camber strain.

it is often considered more art than science. The early uses of heat cambering or bending primarily involved straightening members during the fabrication process. Early papers by Holt (1955), Holt (1965, 1971), Blodgett (1966), and others dealt with distortion and straightening and outlined the basic principles involved in flame or heat cambering. Gradually, as more experience was gained, the process was applied to straightening members damaged in use. The theoretical knowledge of material properties of the member subject to heat straightening or bending was expanded through research by Roeder (1985, 1986) and Avent et al. (2001). Avent authored numerous papers on the effect of heat straightening on material properties, along with a technical guide for the FHWA, *Heat-Straightening Repairs of Damaged Steel Bridges: A Technical Guide and Manual of Practice* (Avent and Mukai, 1998). The paper "What You Should Know about Heat Straightening Repair of Damaged Steel" (Avent and Mukai, 2001) provides a summary of this work. Another summary of heat straightening is "Synthesis Study: Heat Treatment and Its Effects on Rehabilitating Steel Bridges in Indiana" (Lackowski and Varma, 2007).

Heat cambering uses the basic material properties of structural steel to upset portions of the web and bottom flange of a beam resulting in camber. If you know the thermal coefficient of expansion and the variation of the modulus of elasticity with temperature and combine these with a knowledge of the metallurgic phases and a method of applying restraint, it is possible to safely upset a portion of a beam resulting in camber. In Figure 3-7, Roeder shows structural steel will expand proportionally to its temperature up to 1,200°F. Additionally, Roeder shows in Figure 3-8 that the modulus of elasticity will decrease by approximately half at 1,200°F. There are a number of modulus of elasticity curves that vary from this curve depending on testing procedure and the addition of a creep factor. The reduction in modulus of elasticity with heat indicated by this curve is a reasonable guide when planning heat cambering. The yield stress also decreases at a significant rate at temperatures above 800°F as shown in Figure 3-9.

The elastic strain or the amount of deformation that can be applied without causing permanent deformation of the material is shown in Figure 3-10 as a function of temperature for a material that is fully restrained in one axis. The intersection of the plastic flow with the elastic strain is the point at which permanent deformation takes place. While the condition of perfect restraint is not usually present during heat cambering, the figure does indicate how the thermal expansion of the material combined with the reduction in modulus of elasticity and yield strength combine to cause plastic flow or upsetting in the material.

Steel, when heated, tries to expand uniformly in all directions. If it is restrained in one or more directions, according to the Poisson ratio, it will expand more in the free direction. This will result in a thickening of the material transversely if it is restrained longitudinally. When cooled, the material will reduce proportionally in all three dimensions resulting in a reduction in the longitudinal length as illustrated in Figure 3-11.

When straightening members with heat, several different heating patterns are used. These patterns are shown in detail in the FHWA guide *Heat-straightening Repairs of Damaged Steel Bridges: A Technical Guide and Manual of Practice* (Avent and Mukai, 1998) along with recommendations for equipment and procedures for heating. When cambering

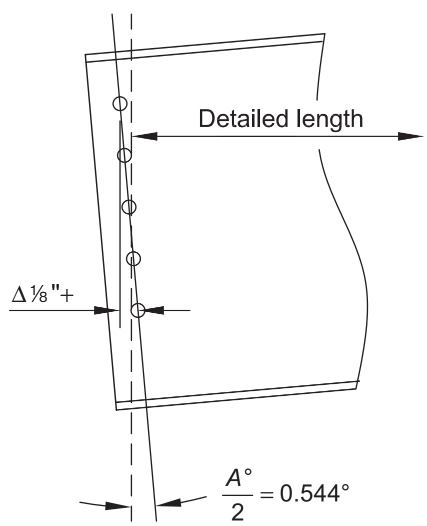


Fig. 3-6. End connection slope.

beams, a vee heat pattern in the web combined with a strip heat in the flange is most commonly used. It is also possible on some lighter members to use only a line-type heat. These patterns will be discussed in more detail in later sections.

The amount of permanent strain that will occur with heat is dependent on the temperature of the heated metal and the restraint that is applied. It is obvious from the previous figures that as the material reaches temperatures near 1,200°F, the amount of permanent strain greatly increases and the yield strength of the material decreases, making the restraint more effective.

The temperature of the heated metal and the rate at which the temperature rises are the most important factors in the heat straightening/cambering process and is one of the most difficult parameters to control. This not only effects the rate of strain, it also effects the restraint. Factors affecting the temperature change include the fuel gas, the size and type of torch orifice, the speed of movement, and the thickness of metal.

AISC *Specification for Structural Steel Buildings* (AISC, 2016b), hereafter referred to as the AISC *Specification*,

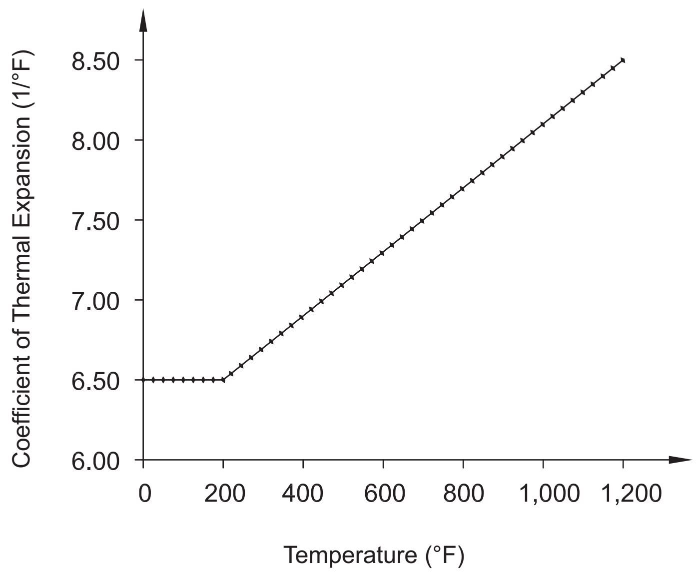


Fig. 3-7. Variation of coefficient of thermal expansion vs. temperature (Roeder, 1986).

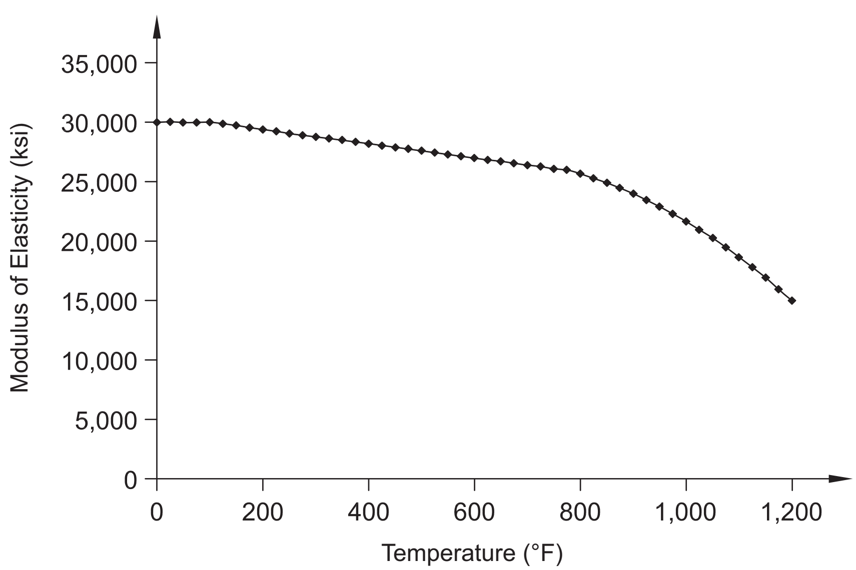


Fig. 3-8. Variation of the modulus of elasticity vs. temperature (Roeder, 1986).

Section M2.1, "Cambering, Curving and Straightening," limits the temperatures of heated areas to 1,200°F for structural steels other than ASTM A514/A514M (ASTM, 2018) and ASTM A852/A852M (ASTM, 2007), which are limited to 1,100°F. The reason for this is that carbon and low-alloy structural steel at temperatures greater than 1,340°F begin to undergo a phase change from a body-centered cubic structure to a face-centered cubic structure. This is known as the lower critical temperature where the ferritic and pearlite crystal structure begins to change to austenite. At the upper critical temperature, around 1,500° to 1,700°F, the change is complete. If the temperature is slowly lowered in a controlled manner, the steel will assume its original molecular shape and properties. The required cooling is difficult to control in the fabrication shop, so most specifications limit the temperature during heat straightening and cambering to temperatures below the lower critical temperature. The temperature can vary significantly across a member as the heat is applied. A measured temperature limit of 1,200°F was set to allow a variation of up to 100°F and still limit the possibility

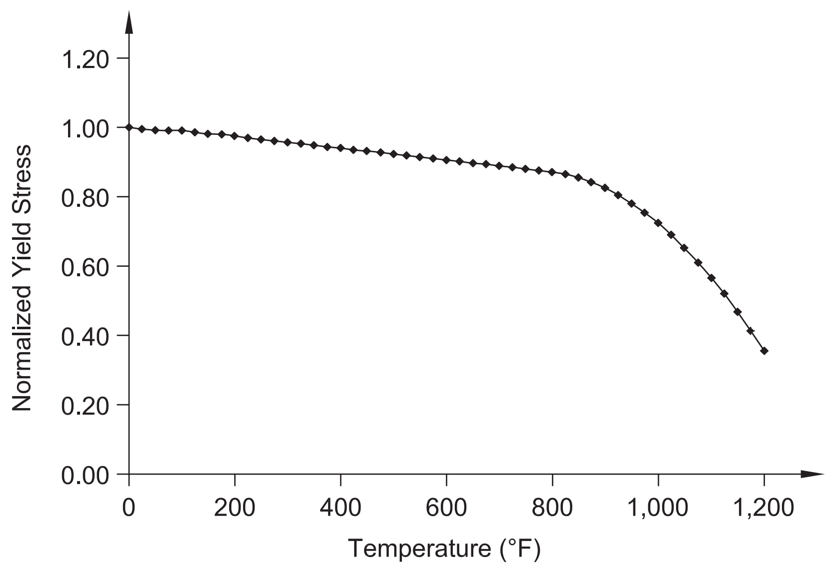


Fig. 3-9. Variation in yield stress vs. temperature (Roeder, 1986).

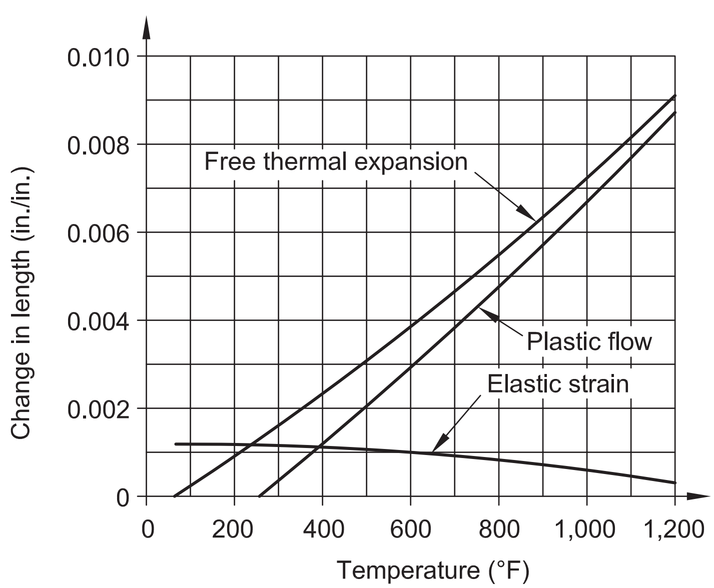


Fig. 3-10. Plastic flow of ASTM A36/A36M confined in one axis (Holt, 1965).

of detrimental material changes due to any local overheating in the process. If the material is accidently heated to a temperature above 1,300°F, surface damage may start to show but, if the member is slowly cooled, the material properties should not change.

3.3.2 Heat Cambering Procedure

The primary restraint in heat cambering comes from the unheated material that surrounds the heated material resisting the expansion. To maximize this restraint, it is important to apply the heat as rapidly as possible in a precise area and manner. Additional restraint may be provided by positioning the member to use the weight of the member along with added load from weights or jacking with special load-limiting jacks.

When heat cambering, the beam is placed with the top flange down and supported as close to the ends as possible to maximize the dead load restraint. Additional weights or a limited jack force may be applied to increase restraint. Special care should be taken to ensure that the entire assembly is stable when the beam starts to move as heat is applied.

A series of heating patterns located near the centerline of the span are used to induce camber. The amount of camber resulting from a heat at the centerline will be twice that from a similar heat at the quarter point. Usually several heats will be required to achieve the design camber, and these can be grouped in the center at 2- or 3-ft spaces, depending on the depth of the member and size of the vee. The space should be large enough so there is adequate room temperature metal adjacent to the heat pattern to restrain the expansion. If the beam is allowed to cool between heats, the heats can be adjacent to each other, or the same area can be reheated as is often done in straightening damaged steel. A typical heat cambering setup is shown in Figure 3-12. Depending on the camber required, heats 1, 2, and 3 would be made, the member allowed to cool, and the camber checked. If more camber is required, additional heats located at 4 and 5 would be applied until the specified camber is achieved.

The typical heating patterns used for cambering a wide-flange beam are shown in Figure 3-13. The vee heat pattern in the web shown in Figure 3-13(a) typically starts at an apex somewhere between the *k* dimension from the lower flange and ⅔ the depth of the member. For heavy members with thick webs, the vee should extend close to the *k* dimension. For members with very thin webs (less than ⅜ in.), it may be necessary to reduce the vee to ½ the depth of the member to prevent out-of-plane distortion of the web. The angle at the apex of the vee will typically vary from 20° to 30° depending on the thickness of the web. The larger the vee, the more movement the heat will produce, but too wide an area may cause the web to distort out-of-plane. The heated area should be limited to what can be heated quickly to limit heat transfer to the adjacent areas and a reduction in restraint. The strip heat used on the flange shown in Figure 3-13(b) should be located directly over the open end of the vee web heat and match the width of the vee where it intersects the flange. When two torches are used for strip heat, the typical torch pattern is shown in Figure 3-13(c).

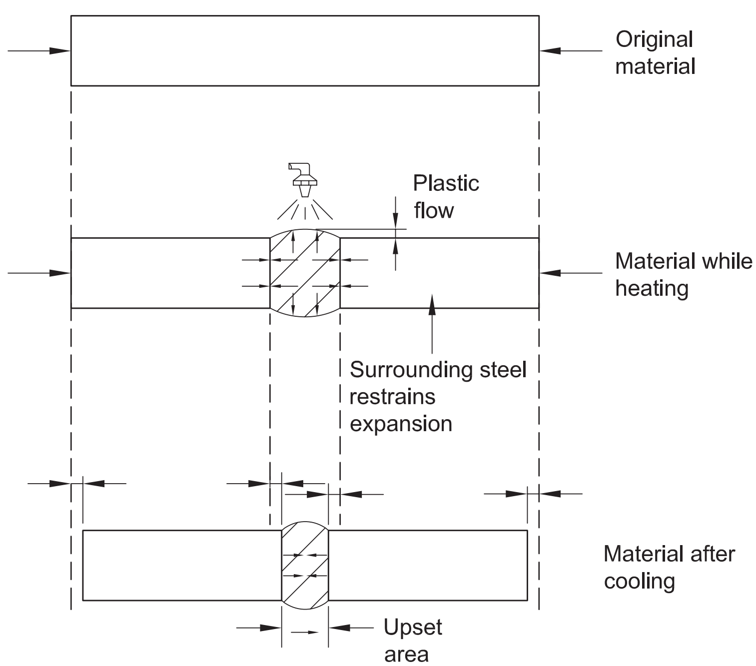


Fig. 3-11. Shortening of steel bar heated while axially restrained.

Table 3-1. Recommended Torch Tips for Various Thicknesses of Material (Avent and Mukai, 1998)		
Steel Thickness, in.	**Orifice Type**	**Size**
< ¼	Single	3
⅜	Single	4
½	Single	5
⅝	Single	7
¾	Single	8
1	Single	8
	Rosebud	3
2	Single	8
	Rosebud	4
3	Rosebud	5
> 4	Rosebud	5

The pattern should be heated as rapidly as possible to limit the transfer of heat to the areas adjacent to the pattern. This will maximize the restraint and increase the movement. A fuel gas with a high heat content, such as propane, or one of the proprietary fuel gases, rather than natural gas, is preferred. The fuel is mixed with oxygen using a heating torch. Selection of the proper type of torch head is important. This can be a single- or multiple-orifice head, and the size will be determined by the type of fuel gas and the thickness of the material. A list of recommended torch sizes from *Heat-Straightening Repairs of Damaged Steel Bridges: A Technical Guide and Manual of Practice* (Avent and Mukai, 1998) is shown in Table 3-1. A No. 8 single-orifice heating tip can be used for most work. It provides a concentrated heat area that can be controlled and will heat rapidly. The larger rosebud type torch heads can be used for 1-in. or greater thicknesses. On these thicker materials, it is recommended that heating be done simultaneously from both sides.

After the size and location of the vee heat is determined, it should be marked on the beam using a 1,200°F temperature heat crayon. In addition to marking the perimeter of the vee, it is recommended that a series of horizontal bands about 3 in. wide should also be marked to aid in controlling the heat application. On heavy members where two torches may be used, both sides of the web should be marked. The strip heat at the top should also be marked in segments.

The heat should be applied starting at the apex of the vee by slowly moving the torch in a circular pattern in each area until the thermal marking starts to melt. The torch should move slowly, progressing across each band, never stopping or moving back over any already heated area. When heating from both sides, it is helpful to number the areas so that the craftsperson can call out the area being worked on. This allows the heating to progress uniformly.

The strip heat on the flange progresses from the center out, but can be done either in a transverse back and forth pattern or a circular longitudinal pattern moving from the centerline out. The transverse pattern is more typically used.

The heated area should be cooled to about 600°F in still air before using dry compressed air to cool to ambient

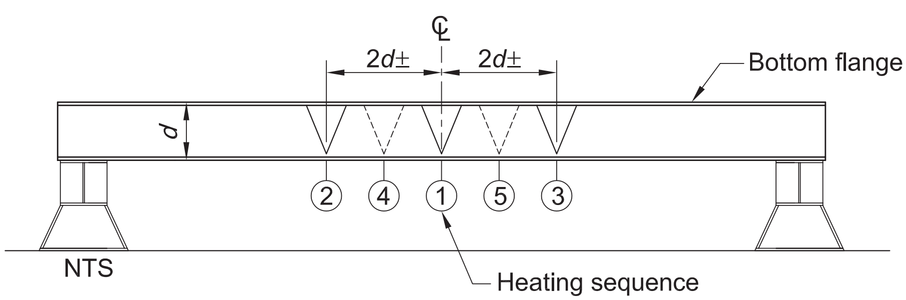
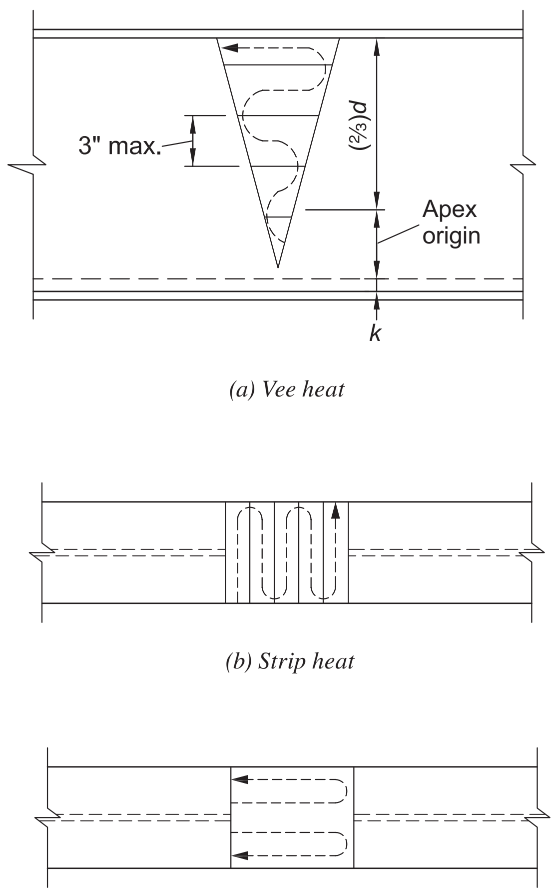


Fig. 3-12. Typical heat cambering setup.

temperature. Studies by Roeder (1985) and Avent et al. (2001) indicate peak residual stresses in both flanges will be in compression and on the heated flange may approach the yield level. The other material properties will not change significantly and thus the beam will perform as designed.

3.3.3 Special Cases

Heat cambering can be used to provide reverse or compound cambers, and special camber profiles if required. Reverse cambers typically require a relatively large change in curvature, which require more heats and will increase the cost. It may be cost effective to provide a larger section to reduce or eliminate the camber required. If deflection of the continuous cantilever is still a concern, provide camber for the cantilever only and avoid the use of reverse camber.



(a) Vee heat

(b) Strip heat

(c) Two torches

Fig. 3-13. Typical heat patterns for cambering wide-flange beams.

Chapter 4
Designing Camber for Composite Beams

The reason most engineers utilize camber in their member designs is to provide the least weight steel member that can support the anticipated loads while still meeting the specific project constraints. This philosophy corresponds to the assumption that least weight also means least cost for the project. There has been some concern in the past that the least weight assumption might be offset by the cost of cambering. The advances in cambering equipment have greatly reduced the cost of cambering. A general guide is a beam weight savings of 3 to 4 lb per lineal foot will offset the cost of mechanically cambering typical floor beams.

It should be stated that a beam that is properly designed for all the anticipated loads should have adequate strength regardless of what percentage of the expected deflections are offset by camber. Introducing camber into a beam is a mechanism to achieve a more economical beam design with respect to the serviceability considerations. A stiffer beam with little or no camber can be just as valid a design solution as a less stiff beam with camber induced. There are typically multiple combinations of beam size and specified camber that can achieve a satisfactory performance with regard to both strength and serviceability; designers have the flexibility to select the combination that is most appropriate for their situation.

When selecting the amount of camber for their beam design, engineers need to understand the variables that influence beam deflection and the methods of placing the concrete slab.

4.1 CAMBER DESIGN VARIABLES

The design of the camber required to produce a level floor is an inexact process due to the many variables that affect the deflection of the beam. As previously noted in Chapter 1, the design objective of cambering a horizontal framing member is to ensure that the deflected shape of the member stays within the desired deflection criteria. For a composite steel floor member, this is typically $L/360$ for the post-composite applied live load and $L/240$ for the net total load, where L is the beam length, although these limits may vary depending on project-specific requirements. There are, however, many variables dependent on engineering judgment that affect the deflection of a beam that make selecting the amount of camber an art rather than a science. Some of these variables, such as the actual restraint provided by the member end connections, are difficult to evaluate precisely. Similarly, while not often considered during the design phase, the method of concrete placement on the deck can also affect the deflection performance of the beams and the levelness of the concrete floors. To complicate matters even further, we cannot simply assign a conservative value because there are potential issues associated with both over-specifying and under-specifying the amount of camber for a given beam size. Fortunately, by examining the different variables and understanding the role each plays in influencing the beam deflections, it is possible to select a camber value for a beam design that will result in an acceptable final product.

4.1.1 Concrete Placement

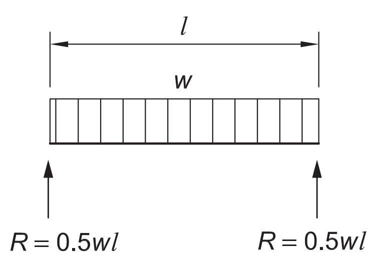
In today's construction industry, most floor slabs supported by steel beams use some type of steel deck as a form. The steel deck manufacturers publish load tables for their products that provide design weights for standard slab thicknesses for each deck configuration based on concrete unit weights for lightweight and normal-weight concrete. These are air-dried weights for calculating design strengths; the wet-concrete weights can be several pounds per square foot heavier. Additionally, the aggregates that are used in the concrete mix vary across the country, causing the dry unit weight of the concrete to vary by as much as $\pm 5\%$ relative to what is shown in the published tables. While these published load tables are useful in estimating the slab self-weight to use in the building design, the authors recommend designers check with concrete suppliers local to a project's location if they are unfamiliar with the concrete weights for that area.

For typical composite slab construction, no negative moment reinforcing is provided in the slab over the beams. As such, it is assumed there will be some minor cracking, controlled by the slab temperature and shrinkage reinforcing, that allows the composite slab to perform as a simple span beam with the steel decking serving the purpose of positive moment reinforcing. In accordance with this assumption, designers typically assume the uniform slab loads are distributed equally to each beam supporting a slab segment, as depicted in Figure 4-1(a). Most steel decks, however, are installed in a multi-span condition. Steel deck manufacturers and steel erectors prefer to install triple-span deck whenever possible, then double-span or single-span only where necessary. Multi-span deck arrangements are more efficient at supporting the wet weight of the concrete during the concrete placement operations and allow the slab to be constructed with a lighter gage deck or to push the deck spans further. As can be seen in Figures 4-1(b) and (c), the reaction to the supports will vary depending on the deck continuity. The reactions in Figure 4-1 are based on rigid supports rather than flexible beam type supports. If we assume the beams have relatively similar stiffnesses, when the interior beams deflect due to the load, some of the difference in load will be transferred to the beams at the deck splice. If the deck layout is known, engineering judgment can be used to adjust the difference in reactions for flexible supports. In the final, post-composite configuration, the loads will redistribute and the loads to the supporting beams will be more consistent with the simple-span configuration. Whether the deck is a single span or multi-span will not affect the analysis or design of the composite beam for ultimate loads.

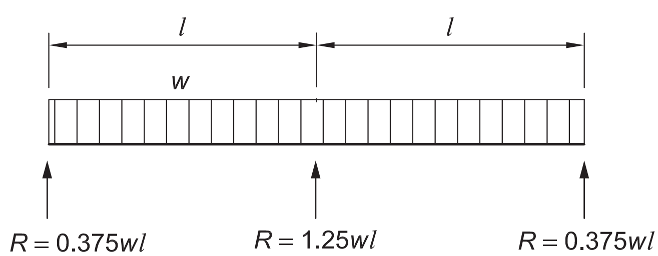
The method specified for screeding of the concrete will also affect the volume and weight of the concrete placed. Practice in the industry is mixed, with some engineers preferring to require the concrete be screeded to provide a uniform plane surface and other engineers preferring to require the concrete be placed to a uniform slab thickness as shown in Figure 4-2. There are pros and cons to both methods that should be considered when selecting the amount of camber for a beam. These issues are discussed in greater detail in Appendix A, Section A.2. When the uniform plane method of screeding the concrete is specified, the beams are typically cambered for less than the calculated deflection due to construction dead load. The deflection calculations should include an allowance for the weight of the extra concrete needed in the middle to provide the required minimum thickness at the supports. Typically, this allowance is based on the anticipated downward deflection under the design concrete thickness. For a detailed method of calculating the additional concrete load, see "Ponding of Concrete Deck Floors" (Ruddy, 1986). While Ruddy does state that deflection of the metal deck could be neglected, there may be cases where slab thickness and spans would require this to be considered.

4.1.2 Connection Restraint

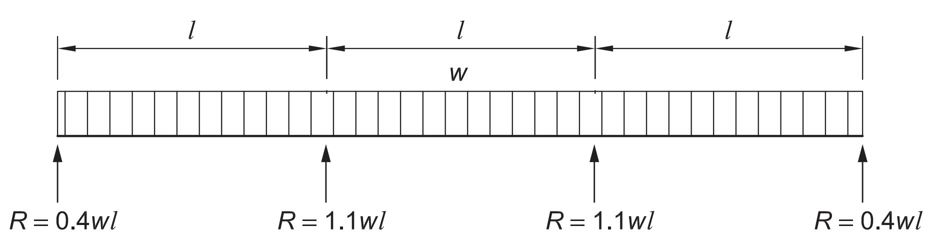
Connection restraint depends on stiffness of the connection, the rotational stiffness of the supporting member, and the geometry of the connection. Typical floor and roof beams that are not part of a lateral frame are designed with simple shear connections, assuming no rotational restraint at the supports. AISC *Specification* Section B3.4a defines a



(a) Simple-span condition



(b) Double-span condition



(c) Triple-span condition

Fig. 4-1. Loading diagrams and support reactions (rigid supports).

simple connection as a connection that "transmits negligible moment" and "may be assumed to allow unrestrained relative rotation between the framing elements being connected." AISC *Specification* Section J1.2 requires that simple connections accommodate rotation of the beam end to be consistent with the assumption of flexibility. During analysis and design of simply supported beams, it is acceptable to idealize simple shear connections as completely flexible. In reality, some rotational restraint will exist. AISC *Manual* Figure 10-1 (reproduced here as Figure 4-3) illustrates the general relationship between a simple shear connection, curve A, and expected rotational restraint. Some restraint is almost invariably going to be present in most standard simple shear connections, but determining the amount of the restraint and its effect on the beam deflection is very difficult to quantify. The simple shear connection curve in Figure 4-3 is nonlinear and indicates the restraint moment does not significantly increase when the beam-end rotation/deflection increases.

By neglecting the connection restraint during beam design and treating the end connections as fully free to rotate, the expected beam deflections will be overestimated. This is generally considered a conservative design assumption as it ensures the designer will select a member size for the beam that can meet the deflection criteria based on the worst case expected beam deflection. However, when trying to balance the amount of deflection with the amount of camber to be imposed, an overestimation of the anticipated beam deflections can result in a beam that still has a positive, upward curvature even after the dead loads are applied.

There have been several studies of simple and/or partially restrained connections (Goverdhan, 1984; Geschwindner, 1991; Ioannides, 1996) that have attempted to quantify the effect of connections on beam deflections. All these studies, however, are based on the face of the connection and the support being in the same vertical plane. Figure 3-6 shows that connections of cambered beams are typically normal to the slope of the beam at the connection face. The studies also assume that the supports are rigid. This assumption is valid for beams framing into the strong axis of columns as shown in Figure 4-4(a) or beams framing into interior girders with balanced loads as shown in Figure 4-4(b). It is not valid for beams framing into spandrel girders as shown in Figure 4-4(c) or girders with large unbalanced loads on either side of the girder as shown in Figure 4-4(d).

Within the connections themselves, short-slotted holes in single-plate connections or in the outstanding leg of single-angle connections are frequently used to accommodate the slope of the beam connection, which may also reduce the restraint that can be expected. The amount of restraint for these connections will vary with the friction capacity of the bolts. The restraint of double-angle connections will vary with the geometry of the connection and the stiffness of the angle.

These factors all combine to make it very difficult to accurately estimate or quantify the magnitude of connection restraint during the beam design process. To account for this unknown contribution of connection restraint on the beam deflection, designers typically specify camber amounts based on engineering judgment that are less than the calculated simple-span deflection from the fully applied dead load. It is reasonable to assume that connections of beams framing into spandrels or girders without balanced framing on the opposite side will have little or no restraint due to the rotation of these members. Connections of beams framing into columns or girders with balanced connections as shown in Figure 4-4(b), however, will have restraint. If we assume that the restraint is a small portion of the fixed-end moment

Fig. 4-2. Power screeding concrete to uniform thickness.

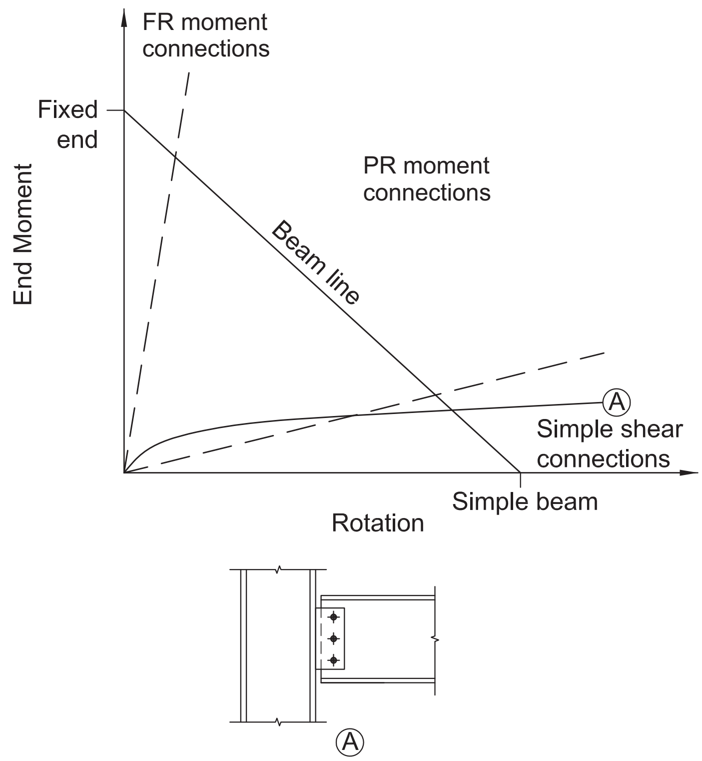


Fig. 4-3. Illustration of typical moment rotation curve for simple shear connection.

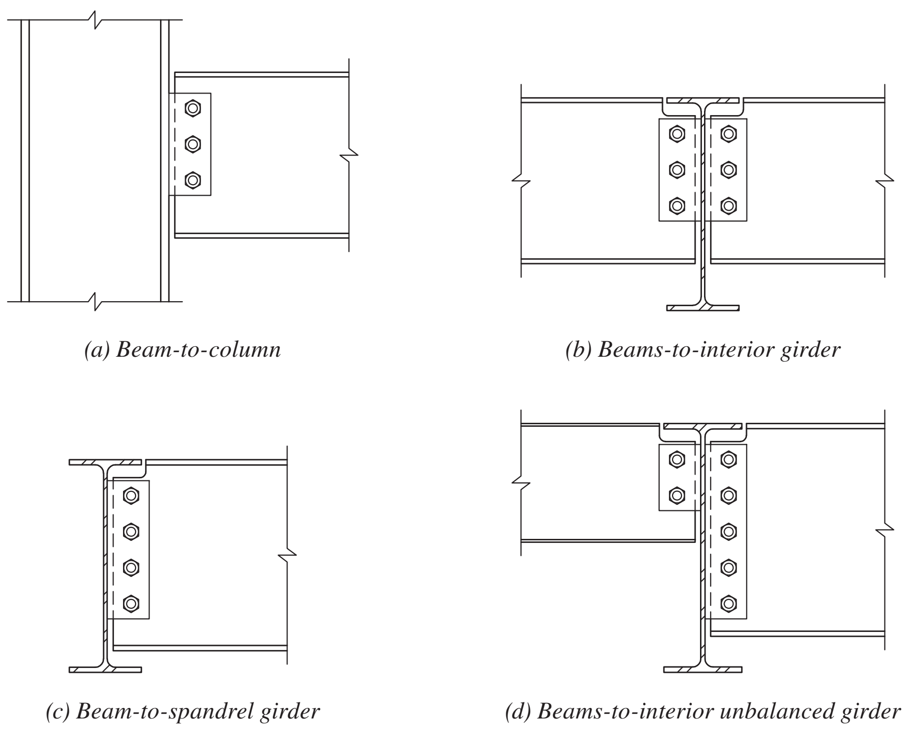


Fig. 4-4. Connection conditions.

restraint, it is possible to approximate the restraint effect on the calculated simple beam deflection. A reduction of simple span deflection of 5 to 10% for one end restrained and 10 to 15% for both ends restrained seems reasonable. These are only approximate values; the actual restraint will vary with the specific connection details and the amount of deflection of the member.

4.1.3 Welded Attachments

Spandrel beams and similar members with field-welded attachments may lose some camber depending on the weld details. Use of minimum-size stitch welds that are not closely spaced will limit the negative heat effects that could reduce, or relax, the imposed camber. For example, the small amount of welding required to fasten a bent-plate edge form typically has a negligible impact on the final beam camber. However, for a built-up member with more significant welding required to make two or more steel shapes act together, the amount of welding could be significant enough to cause a measurable loss in camber. Built-up members should, where possible, be shop fabricated so that any out-of-tolerance can be corrected.

There has been limited study of the effect of shear stud welding on loss of camber. Fabricators know from experience that welding steel headed stud anchors to light members such as long embedded angles can cause distortion. Welding steel headed stud anchors to the top flange of a beam is similar to applying a heat spot. A study by Lange and Grages (2009) on the deflection behavior of cambered beams investigated the effect of size and spacing of typical steel headed studs welded on different Eurocode beam profiles. The calculated loss of camber using Lange and Grage's proposed equation for the W21×44 example with the 40-ft span in Chapter 2 is a loss of camber less than ⅛ in. It is theoretically possible that for light members with a large span/depth ratio and a large number of steel headed stud anchors (high percentage of composite action), welding of the stud anchors could cause a loss in camber; otherwise, the loss is too small to be considered in typical designs.

4.1.4 Fabrication Tolerance

As mentioned in Chapter 3, AISC *Code of Standard Practice* Section 6 identifies permissible tolerances for camber resulting from the fabrication process. For beams less than 50 ft in length, the camber may vary by –0 in. to +½ in. from what is specified, and for beams exceeding 50 ft, the tolerance is an additional +⅛ in. per 10 ft of beam length. From a practical perspective, utilizing modern-day machinery to impose the camber makes it easy to achieve these tolerances in most cases. While theoretically two adjacent beams could vary in camber by ½ in. when loaded, the resulting deflections will tend to equalize, provided the restraint conditions are similar.

The AISC *Code of Standard Practice* camber tolerances only allow for fabricated camber to be equal to or greater than what is specified. The result is that most members fabricated with camber will have more camber than was specified. The camber study discussed in Section 2.3 (Larson and Huzzard, 2003) indicated that beams that were cambered to AISC *Code of Standard Practice* allowable tolerances at the mill arrived at the fabrication shop with an average of 3⁄16 in. more camber than specified in the design. Because the fabrication process results in beams that are likely over cambered, engineers should take this into consideration when specifying beam camber.

4.1.5 Concrete Shrinkage

Concrete shrinkage is typically not included in camber calculations because it occurs after the beam has achieved its composite strength level. It can be a serviceability concern, though, especially on larger span-to-depth ratio beams. The shrinkage of the concrete results in a positive moment in the beam and increases the service load deflection. AISC *Specification* Commentary Section I3.2 provides some guidance on shrinkage effects; see Figure 4-5. Studies by Viest et al. (1997), Leon (1990), Leon and Alsamsam (1993), and Kim (2014) indicate that accurate predictions are difficult because the effects vary with the slab reinforcement and the type of aggregate. The majority of the shrinkage effect occurs within the first 40 days, so it might be a design consideration for certain spans and loadings. The Commentary also suggests that long-term deformation due to shrinkage can be calculated using the simplified model shown in Figure 4-5. The effect of the shrinkage is taken as an equivalent set of end moments given by the shrinkage force (long-term restrained

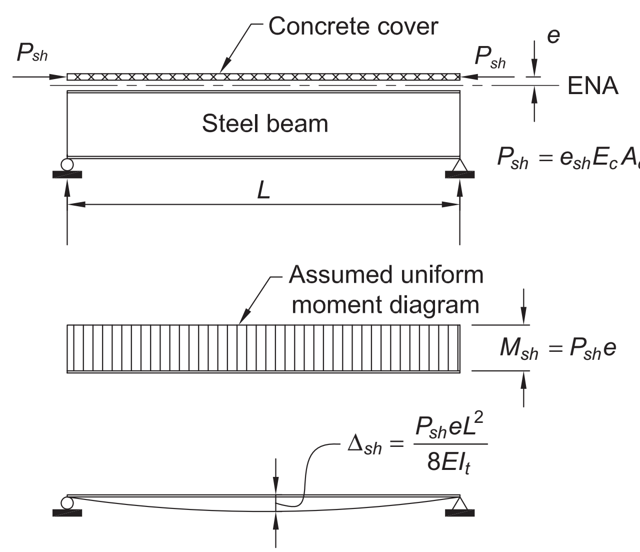


Fig. 4-5. Calculation of shrinkage effects (AISC, 2016b).

shrinkage strain multiplied by the modulus of concrete multiplied by the effective area of the concrete) multiplied by the eccentricity between the center of the slab and the elastic neutral axis. If the restrained shrinkage coefficient for the aggregate is not known, the shrinkage strain, e_{sh}, for these calculations may be taken as 0.02%.

4.1.6 Material Properties

Residual stresses have been shown not to be a problem in either loss of camber in shipment or in the ultimate strength capacity of the composite section. They may, however, contribute slightly to the deflection of the beam at full service loads. The variations in residual stress depend on mill rolling stresses and the camber and cambering process. Because this effect will be small, there has been no effort to quantify it.

Strain softening has been cited in one paper (Lange and Grages, 2009) for its influence on the deflection behavior of composite beams. When members that have been strained to yield in one direction by cambering and then unloaded and finally loaded in the reverse direction, the stress-strain curve varies from a straight line to a slightly rounded curve. This is called the Bauschinger effect; at larger loads deformation increases. Figure 4-6 shows that when the load is reversed, the yield stress, σ'_s, is less than the original yield stress when first loaded, σ_s. This effect occurs only as the loads approach yield. For camber design loads where the stresses have been reduced by a safety factor, the Bauschinger effect is typically not considered.

4.1.7 Span at Columns

When floor beams at the gridline frame to the column flange, the member size does not typically change, but the reduction in effective span length can be significant. Assuming W14 columns and a 40-ft span, there would be approximately a 9% reduction in deflection due to reduced span length. The reduced deflection of members spanning to columns relative to those that frame to beams should be considered when specifying camber.

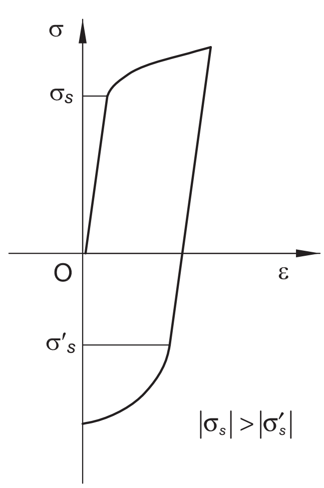


Fig. 4-6. Bauschinger effect (Lange and Grages, 2009).

4.2 CALCULATING PRECOMPOSITE BEAM DEFLECTIONS

A survey across the country indicates most designers when calculating beam deflections will use the beam weight and the concrete load from the appropriate deck catalog on the tributary area and apply the equation for a uniformly loaded, simple-span beam. A review of the design variables mentioned previously can be helpful in determining what, if any, adjustment in the calculated deflection might be required when using this approximate method to calculate deflection.

We can start by neglecting the effects of any welded attachments, material properties, and, in most cases, concrete shrinkage effects. The fabrication tolerance effect should be considered when specifying the camber, but it does not affect the deflection calculation. The deflection of interior beams should slightly increase because of the increased load due to deck continuity and the possible increase in the wet unit weight of the concrete. There will also be a decrease in deflection due to connection restraint at the girder. It is reasonable to assume that for typical interior beams, the increase in load due to deck span and the decrease due to connection restraint tend to offset each other, and little adjustment in the simple-span deflection calculation is required.

The deflection of beams framing to columns, however, will see a significant decrease in deflection due to the deck span effect and more connection restraint along with a span length reduction where appropriate. If the member framing to the column is the same size as the typical beams framing to spandrels, the deflection at the column line may only be 75 to 90% of the deflection of beams that frame to spandrel beams.

4.3 LOAD TO OFFSET

One of the most important decisions an engineer must make in selecting the amount of camber to specify for a beam is how much of the deflection due to the imposed loads they want to offset. The authors surveyed several designers from offices around the country, and most of the practicing engineers surveyed responded that they offset 75 to 85% of the precomposite simple-span dead load deflection when selecting the magnitude of beam camber with little deviation from this rule of thumb. These designs are based on the concept that the concrete would be screeded to a uniform elevation. The goal of this reduction in camber is to compensate for the plus fabrication tolerance and any connection restraint, but still provide a net downward deflection. Firms that specified constant thickness slabs specified cambers up to 90% of the calculated simple-span dead load deflection, the intent being to compensate for the plus camber tolerance and have a slab that is approximately level.

For both methods, the camber specified for beams at column lines varied from using the same camber as interior spans to using 70% of the interior span camber. Camber for beams framing into columns should be based on the reduction in simple-span deflections at columns lines discussed previously, along with the effect of added connection restraint. When screeding to a uniform elevation, specifying the same camber at column lines as that specified for interior beams will result in an increase in concrete volume at the interior beams to compensate for the high points at the column line beams. When screeding to a uniform thickness, specifying the same camber as the interior beams will result in noticeable high points at the column lines.

Few designers specify camber for the full dead-load deflection because of the uncertainty of the actual camber due to the variables discussed previously and the fabrication tolerance, with the exception of using the constant slab thickness method when the goal is to provide positive camber to compensate for any concrete shrinkage and/or a portion of the superimposed dead load. A procedure for designing floors with a positive camber can be found in "Camber—An Art and a Science" (Lederle, 2003). This camber design procedure uses the full precomposite dead load, adds approximately half of the post-composite dead load, and specifies the floor to be screeded to a constant thickness. It is important to note that when using this method, the camber specified for the beams at the column lines is about 75% of the camber specified for the typical interior beams.

While the goal when screeding to a uniform elevation is for the floor beam to be close to level once the precomposite dead load is imposed, the expectation is that there will be some net negative deflection that is compensated for when the concrete is placed level. If a floor beam is cambered more than the amount of deflection experienced when the wet concrete is placed on the deck, the beam will still retain a slight upward, or positive, curvature. This is typically acceptable when screeding to a uniform thickness, but when screeding to a uniform elevation, it may affect the floor finishing requirements and the design requirements. This will be discussed in greater detail in Appendix A, Floor Levelness.

The practice of offsetting 80% of the expected dead load—referred to here as the 80% rule—has been applied to a broad spectrum of projects and, in most cases, appears to result in typical interior beams that perform well for "average" projects. However, it might not be the best option for all structures. A different percentage of dead load deflection to offset may be more appropriate for thin, lightweight slabs or thick, heavy slabs. Consider the beam deflections resulting from three different hypothetical slab systems where calculated deflections vary from 1 in. to 2 in. to 3 in., respectively. Using the 80% rule and rounding down in ¼-in. increments would result in camber reductions of ¼ in., ½ in., and ¾ in. The ¼-in. reduction might result in over-cambering if the actual camber is +½ in. over the specified camber. The ¾-in. reduction might result in significant extra concrete, resulting in additional deflections and even more concrete.

Anecdotally, engineers and contractors have reported instances when beams supporting thin slabs have not fully deflected during construction and the beam remains curved upward after the slab has been placed. Problems with steel headed stud anchors projecting above the slab surface for thin-slab systems have been reported when screeding to constant elevation. Because of this known history, an engineer designing a constant elevation composite slab using a thin, light slab might consider reducing the specified camber to a smaller percentage of the calculated dead load deflection, thereby increasing the probability that the beam will not retain any camber once the slab is placed. This is especially important for members at the column lines where deflection is smaller. Conversely, when specifying camber for slabs with large calculated dead load deflections, increasing the percentage of load offset will reduce the amount of extra concrete required when screeding to a uniform elevation.

The problem of steel headed stud anchors projecting above the slab is generally only an issue for thin slabs. For thicker slabs, there is sufficient length between the minimum and maximum stud length permitted by the AISC *Specification* that a length can be specified that will allow extra tolerance for concrete cover.

4.3.1 Compounded Deflections

A typical framing bay consists of beams connected to girders that connect to columns, as shown in Figure 4-7. The beams and girders are typically designed as simple-span members unless they are also part of the lateral load-resisting system. During design and analysis of each member, the member deflections are evaluated relative to the member end supports, assuming the member ends are restrained from vertical movement. However, the total displacement of each member will also be dependent on whether the framing supporting that member is also free to displace. For a beam framing into a girder, the total beam displacement is a superposition of the beam deflections on top of the girder deflections. This superposition of deflections of multiple members is referred to as compounded deflections.

When calculating precomposite dead load deflections, the amount of extra concrete that will be required to maintain a constant floor elevation will correspond to the total floor displacement as depicted in Figure 4-8, not just the individual member displacements. A ponding analogy can be used for analytically estimating the quantity of additional concrete that will be required for a level floor (Ruddy, 1986).

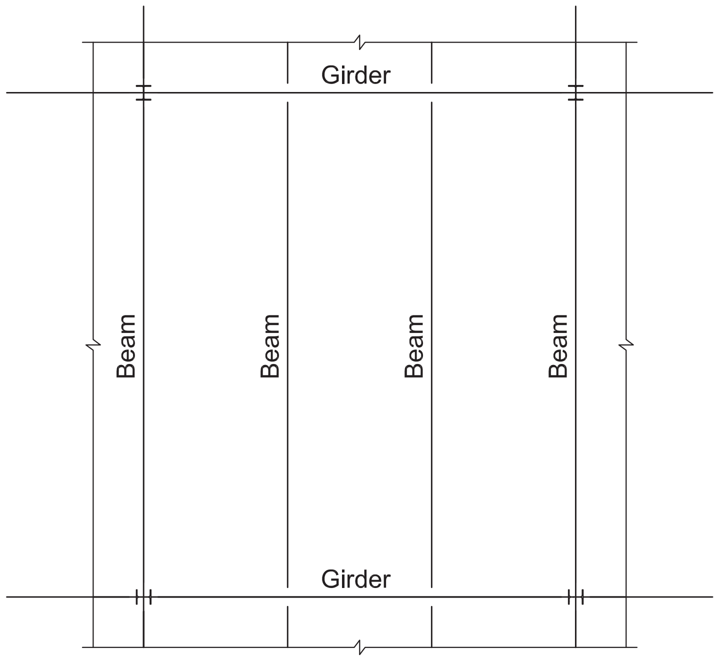


Fig. 4-7. Typical framing bay.

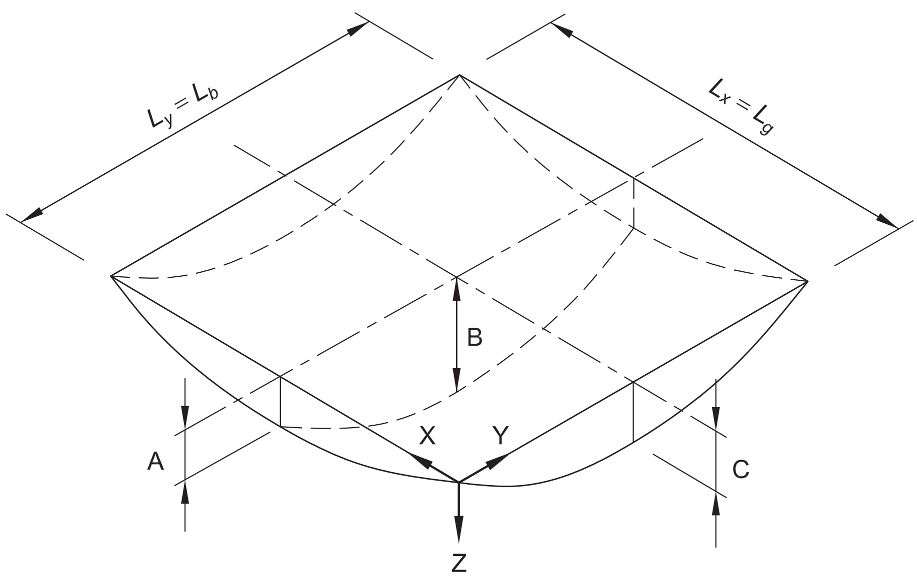
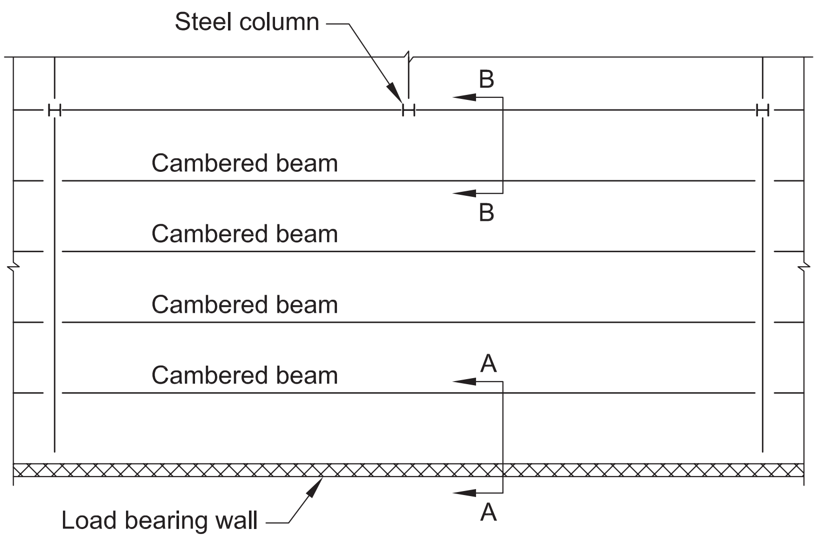


Fig. 4-8. Single bay slab deflection.

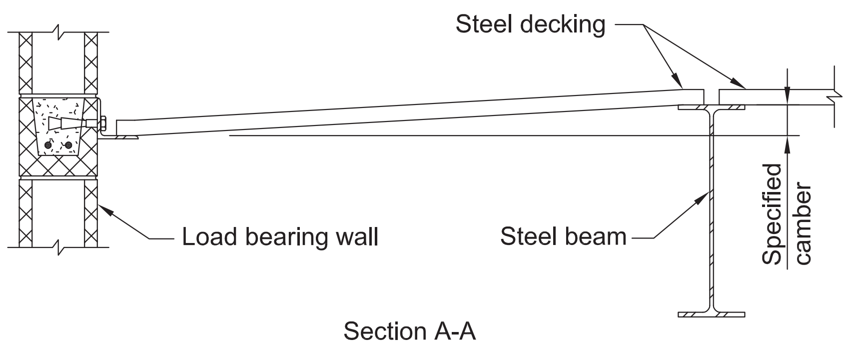
4.3.2 Floor Plan Framing Considerations

Abrupt changes in the framing layout or stiffness between adjacent members can create conditions where specifying camber results in constructability challenges. The differences in the top of steel elevations before any load is applied to the members may not be able to be accommodated by the metal deck, especially if the cambers are large or the deck design is dependent on multi-span deck installation.

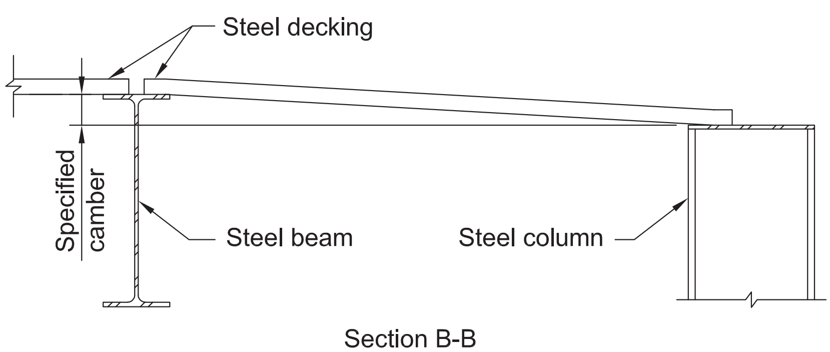
Consider the partial framing plan shown in Figure 4-9(a) and the associated sections shown in Figures 4-9(b) and 4-9(c). Once all the loads are applied to the framing, the estimated top of steel for the cambered beams, steel columns, and deck support at the load bearing wall should all be nominally the same. However, during construction the elevation of the cambered steel beams at midspan can be significantly higher than the adjacent framing. At these conditions, the deck may not be able to flex enough to achieve a satisfactory connection to the lower element. It may be possible to install a single span of deck and warp the ends at the deck bearing if the specified camber relative to the deck span is not too large as shown in the details. Alternatively, it may be more appropriate to install a beam adjacent to the column line that requires only half of the typical specified camber.



(a) Partial framing plan



(b) Section at load bearing wall



(c) Section at mid-span column

Fig. 4-9. Floor plan framing considerations.

Chapter 5
Camber for Special Conditions

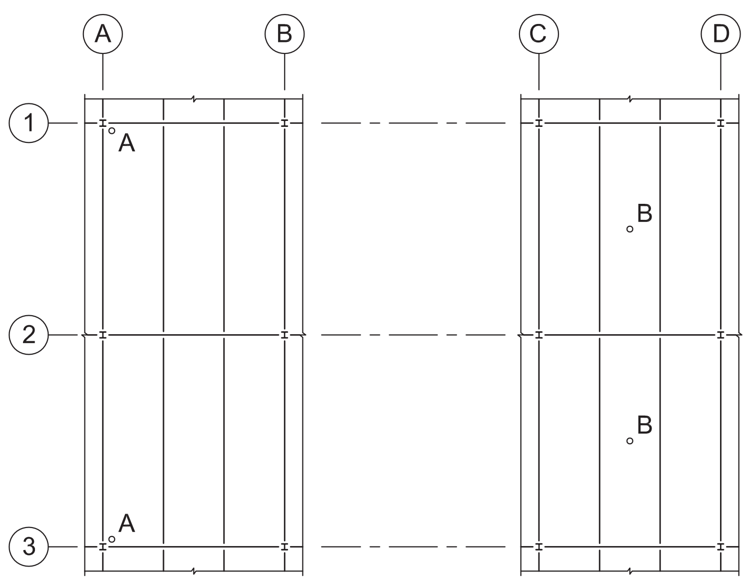
Camber may be specified for members other than composite beams. Although for some of these members the method of cambering may be similar to that for composite beams, there may be other methods or alternatives to cambering that the designer should consider.

5.1 ROOF MEMBERS

When considering roof member design, the criteria for selecting camber is slightly different. The design goal is still to offset the expected deflections due to the dead load, but it is common to also offset 100% of the superimposed live loads. This offset can be accomplished by camber, by providing enough slope to maintain positive drainage, or by a combination of these methods. Camber is typically required when there are large dead loads or very long spans. In deciding how much camber to specify for a roof beam, designers should consider the roof drainage and the roof slopes. Figure 5-1 illustrates two different, yet common, drainage configurations.

In Figure 5-1(a), drains are shown at the two A points near the columns at grids A-1 and A-3. With a layout like this, the roof steel will frequently be sloped with low points along grids 1 and 3 and a high point along grid 2. However, even if the steel is installed at a constant elevation and the roof drainage is attained through built-up roofing materials, the concept is the same. A nominally flat roof will typically be pitched with a minimum slope of ¼-in. per foot from the high point to the low point. In this scenario, it is reasonable to select beam camber that offsets the combined structure self-weight plus 100% of the expected superimposed dead load. In the final installed condition, the beam may still have a positive curvature, but that is not detrimental to the structural performance and, unless it is unusually excessive, should not impact the roof drainage, which is the main serviceability consideration for the roof.

In Figure 5-1(b), drains are shown at the two B points located in the center of each bay. When the drains are not located near columns, the framing is typically installed level with a constant specified top of steel elevation for the framing. If the beams in this scenario retain a positive camber once the structure self-weight and the superimposed dead loads are all present, then the low point in the roofing surface will occur at the beam supports along grids 1, 2, and 3 away from the drains. To avoid creating a situation where ponding could occur due purely to the steel design, camber specified for members configured in this manner should be selected to allow for a net negative curvature once all the dead loads are applied; only a fraction of the total dead load



(a) Drainage at column lines *(b) Drainage at mid-bay*

Fig. 5-1. Typical roof framing bay.

plus superimposed dead loads should be offset. That percentage is an engineering judgment decision that should take into consideration the magnitude of the loads, the member spans, and the net total deflection anticipated.

5.2 TRANSFER GIRDERS

Transfer girders are typically noncomposite heavy members designed to support large concentrated loads from columns that may support multiple floors. Transfer girders are typically very conservatively designed to limit both dead load deflections and some live load deflections. The girder will not see all the dead load deflection until the steel and concrete is placed on all of the floors that are supported. The designer will have to specify the camber that is expected to remain in the girder as each floor is placed and adjust concrete elevations as necessary.

Transfer girders may be built-up plate girders or heavy rolled shapes possibly with cover plates. Welded plate girders can be cambered by cutting the web plate to the specified camber curve and then fitting the flanges to the web. The camber should be checked after welding to verify there has been no change after fit-up. Camber adjustments can be made by planning the welding sequence. Welding the bottom flange first will tend to add positive camber. Heat cambering can also be used to correct any small camber change. Rolled wide-flange sections can be cambered using mechanical cambering or heat cambering as described in Chapter 3, depending on the fabricator's equipment capacity. Cover-plated wide-flange sections should be cambered before welding the cover plates.

A cost effective alternate when using either built-up or heavy rolled sections that the fabricator may want to consider is to detail the girder without camber and adjust the column length and floor beam connections to the required camber elevations.

5.3 CANTILEVER BEAMS

Cantilever beams and the cantilever portion of continuous beams will be noncomposite and must be designed for the deflections of both dead and live loads. These members are typically sized stiffly enough that camber is not required. Long cantilever members, however, may be sloped and have a preset elevation specified for the free end. The AISC *Code of Standard Practice* Section 3.1 requires that "Structural Design Documents shall clearly show or note the work that is to be performed" and one of the items listed is: "The preset elevation requirements, if any, at free ends of cantilevered members relative to their fixed-end elevations." When the cantilever beam has a moment connection at its support, this connection should be designed with enough adjustment that the free end of the beam can be supported at the specified preset elevation before being fixed. It is important that the load line not be released until the cantilever is safely shored or an adequate connection is made. The AISC *Code of Standard Practice* Section 7.13.1.2(e) specifies the tolerance for the cantilever "is equal to or less than $1/500$ of the distance from the working point at the free end," and the Commentary states, "This tolerance is evaluated after the fixed end condition is sufficient to stabilize the cantilever and before the temporary support is removed."

Cambering of cantilever beams that are continuous over the support should be avoided. The reverse cambers are especially difficult to fabricate and should be avoided because the radius of curvature required is typically much smaller than the typical camber curve radius. These types of members usually require costly heat cambering. When camber is specified for the cantilever portion of a continuous member, the camber is checked in the shop using the camber diagram rather than using a preset elevation in the field.

5.4 TRUSSES

The types of trusses and their uses are so varied that it is beyond the scope of this document to adequately discuss all of the camber requirements and the methods used to provide camber. The amount of camber to specify is a serviceability issue and varies from cambering for dead load only or inclusion of partial or full live load. The advancement in analysis software has simplified the calculation of deflection; however, in most cases there will still be assumptions on connection stiffness, support restraint, and actual loads so that the deflection will be a design approximation.

The AISC *Code of Standard Practice* requirements for truss camber tolerances are as follows:

> **Section 6.4.5.** For fabricated trusses that are specified in the contract documents with camber, the variation in camber at each specified camber point shall be equal to or less than plus or minus $1/800$ of the distance to that point from the nearest point of support. For the purpose of inspection, camber shall be measured in the fabricator's shop in the unstressed condition. For fabricated trusses that are specified in the contract documents without indication of camber, the foregoing requirements shall be applied at each panel point of the truss with a zero camber ordinate.

Section 6.4.5 only addresses trusses that are assembled in the shop as shown in Figure 5-2. Advances in fabrication technology—such as computer modeling with downloads to computer-controlled shop equipment, along with the use of slip-critical bolted connections—have made it possible to fabricate the individual members for large trusses and ship them without costly shop assembly. When these trusses are field assembled, the camber is checked in the unstressed condition in either the laydown position or when fully shored. Trusses that are fabricated in sections and field assembled have the sections checked separately in the shop and the camber rechecked in the field when spliced.

AISC *Code of Standard Practice* Section 7.13.1.2 covers erection tolerances for members other than column shipping pieces. The following paragraphs address truss erection tolerances:

> (d) For a member that consists of an individual, straight shipping piece and that is a segment of a field assembled unit containing field splices between points of support, the plumbness, elevation and alignment shall be acceptable if the angular variation, vertically and horizontally, of the working line from a straight line between points of support is equal to or less than 1⁄500 of the distance between working points.
>
> (g) For a member that is fully assembled in the field in an unstressed condition, the same tolerances shall apply as if fully assembled in the shop.
>
> (h) For a member that is field-assembled, element-by-element, in place, temporary support shall be used or an alternative erection plan shall be submitted to the owner's designated representatives for design and construction. The tolerance in Section 7.13.1.2(d) shall be met in the supported condition with working points taken at the point(s) of temporary support.

Figure 5-3 shows a truss assembled in accordance with the provisions of AISC *Code of Standard Practice* Section 7.13.1.2 (g) in the unstressed condition, which is the typical method used when space is available.

Figure 5-4 shows trusses being assembled in the stressed position while supported by special fixtures using the provisions of AISC *Code of Standard Practice* Section 7.13.1.2(h). This method is used when there is limited space for assembly and/or multiple trusses that make the special fixtures cost effective.

Figure 5-5 shows large, extra-long-span trusses being erected in place on shoring using the provisions of AISC *Code of Standard Practice* Section 7.13.1.2. A special engineered erection plan was used to control the deflection as erection progressed across the span.

Where truss camber is required, the camber diagram is usually specified as a parabolic curve. For trusses with wide-flange chords, the curve is replicated by designating the camber elevations required at specific panel points. The wide-flange chords are fabricated as straight members and sloped as required to the splices at the camber points. The length of the diagonals are adjusted as required to match the slope.

Lighter shop-welded trusses with WT chords, angle chords, and HSS chords are curved as required by using heat or by jacking lighter chords in a fixture to the required curve. Where the diagonals are shop-welded double angles, the lengths can be adjusted when fitting up the truss. It is important when designing cambered trusses with HSS chords and direct-welded HSS web members as shown in Figure 5-6 that gapped type joints are specified. This provides some room for the fabricator when fitting up diagonals to adjust for length.

5.5 JOISTS, JOIST GIRDERS, AND COMPOSITE JOISTS

It is important to be aware that all open-web steel joists and joist girders produced in accordance with the Steel Joist Institute (SJI) *Standard Specification for K-Series, LH-Series, and DLH-Series Open Web Steel Joists and for Joist Girders* (SJI, 2015) are provided with standard camber unless the specifying professional indicates otherwise. When joists or joist girders span 100 ft or less, the approximate fabricated standard camber provided is based on a radius of 3,600 ft. See SJI Table 4.6-1 (SJI, 2015), shown here as Table 5-1.

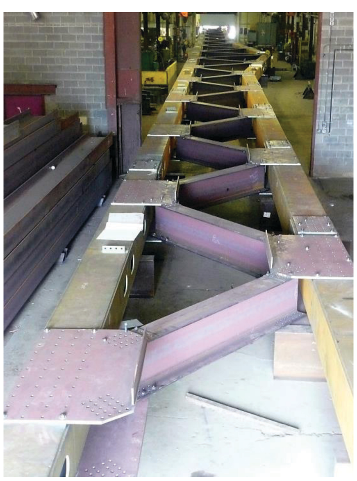


Fig. 5-2. Truss shop assembled to check fit-up and camber.

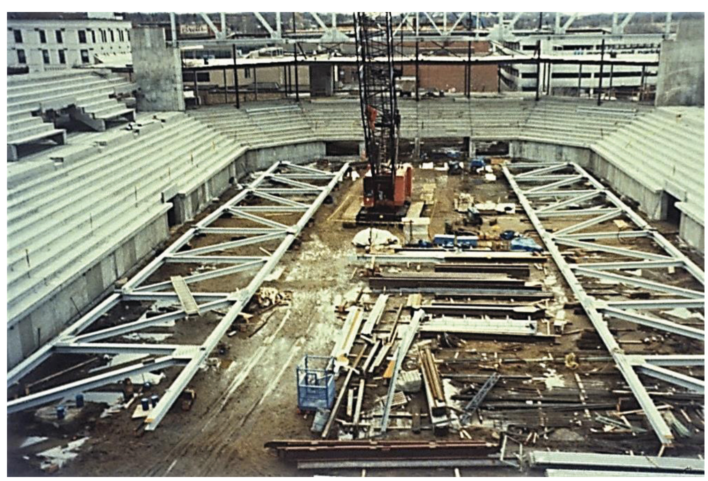


Fig. 5-3. Field-assembled truss in the unstressed condition.

For lengths that exceed 100 ft, the camber provided is approximately span/300. For either condition, this approximate camber is fabricated into the joist or joist girder during placement of the chord and web members while in a rigging table. The camber measured after the joists are erected vertically will be less than the camber provided while in a rigging table because deflection is already taking place due to the self-weight of the joist, other dead load present, the span, and the joist stiffness based on configuration and member sizes.

The user note in Table 5-1 cautions about special considerations for camber that may be needed where the joist elevations are to match up with adjacent framing or a wall. Steel deck is often sized based on a three-span condition, so the elevations may need to transition so the steel deck erector can properly place the steel deck. The camber specified should consider the relative deflections that will take place during later design load application. For double-pitched top chord profiles with steeper pitch rates—that is, greater than 2 in./ft—or for an arched top chord profile, it may be desired to specify no camber where there is a need to match elevation with adjacent framing or a wall due to pitch rate or profile.

The SJI *Code of Standard Practice for CJ-Series Composite Steel Joists* (SJI, 2016), Section 2.6(b), states the following:

> It is standard practice that the CJ-Series joists are furnished with sufficient camber for 100 percent of the non-composite dead load (joist, bridging, deck, and concrete slab). Joist bearings act as pinned/pinned-end

Fig. 5-4. Truss field assembled in supported condition—checking camber.

Fig. 5-5. Trusses field assembled in place using an alternate erection plan.

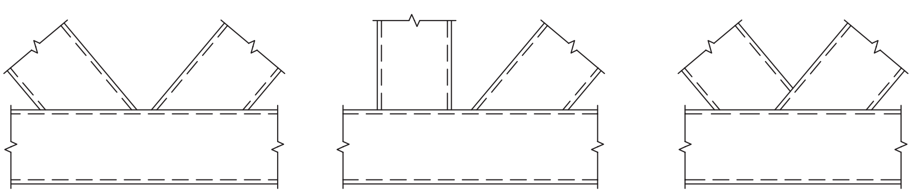


Fig. 5-6. HSS-to-HSS truss connections.

Table 5–1. Fabricated Standard Camber (SJI, 2015)	
Top Chord Length	**Approximate Camber, in.**
20'-0"	¼
30'-0"	⅜
40'-0"	⅝
50'-0"	1
60'-0"	1½
70'-0"	2
80'-0"	2¾
90'-0"	3½
100'-0"	4¼
For lengths exceeding 100'-0", manufactured camber equal to span/300 shall be used. User Note: The specifying professional shall give consideration to coordinating this approximate camber with adjacent framing.	

connections with negligible end rotation restraint provided. Hence one will obtain 100 percent (100%) of the predicted non-composite joist deflection when the full non-composite dead load has been placed on the composite steel joist. With the composite steel joist cambered for 100 percent (100%) of the non-composite dead load and the floor slab placed to a uniform thickness as suggested in Section 9, Concrete Placement, the floor shall be approximately level after the concrete has been placed.

Should the specifying professional strive to achieve a level floor after the composite dead and live loads are placed on the floor, joist camber can be specified on the "Required Design Parameters," see Appendix A. It is typical that the actual composite dead load and live loads supported by the composite steel joist are less than the full design composite dead and live loads.

The designer should be aware that like trusses, calculating the camber for joists, joist girders, and composite joists requires some assumptions on connection and member stiffness. This can result in the actual deflection varying from the calculated deflection.

5.6 CRANE GIRDERS

The deflection of crane girders varies with the crane lifted load and the crane's location on the span. Girders should be designed to limit deflections to industry standards; see AISC Design Guide 7, *Industrial Building Design* (Fisher, 2019). When using built-up sections such as a wide-flange beam with a cap channel, care should be taken to correct any negative camber that might occur due to welding the channel to the beam.

5.7 MEMBERS OF LATERAL LOAD-RESISTING SYSTEMS

Beams or girders that are part of a lateral load-resisting system should not be cambered. For both moment frames and braced frames, the special connection details required for horizontal members connecting to columns are typically large and very stiff. To achieve proper fit-up most economically, the members should intersect at a 90° angle. As was shown previously in Figure 3-6, the end connection of a cambered beam will be sloped relative to the supporting member. Because camber is typically induced into a member after the cutting and machine-based fabrication is completed, the geometry of the connection will vary and affect the fit-up of the member.

5.8 SPANDREL MEMBERS

Spandrel members are beams or girders parallel to the exterior wall that support the perimeter edge of slab of a floor system. Because the exterior building façade system is attached to and/or supported by the spandrel members, the deflection characteristics of spandrel members can have significant impact on nonstructural components.

The typical sequence of construction for a floor is to erect the steel, install the metal decking, and cast the concrete for the floor slab prior to attaching any of the nonsteel façade elements. Nonstructural steel façade framing elements that are installed after the floor slab concrete has been placed and camber has "come out," such as cold-formed metal wall studs or mullions for window wall systems, should not be impacted by whether or not a beam is cambered. For spandrel beams in buildings with these types of façade systems, there can be an advantage to specifying a spandrel beam with camber.

Steel façade support elements such as suspended brick-relieving angles above ribbon windows, as shown in Figure 5-7, are often installed before the floor slab concrete is placed. For these types of structures, the beams should not be cambered. Spandrel beams with supplemental steel framing to support the façade need to be carefully coordinated with the architectural façade details and the façade system joint limitations. Often, especially when considering a brittle façade material such as masonry, the spandrel beams need to be designed for a more stringent deflection criterion than the typical interior beam or girder.

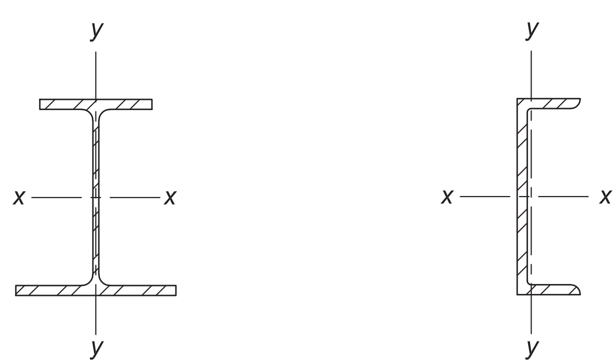
5.9 MEMBERS WITH NONUNIFORM CROSS SECTIONS

Camber is difficult to achieve for members with nonuniform cross sections. While there are many possible geometries of nonuniform cross sections, Figure 5-8 shows samples of two representative conditions—unsymmetrical about the *y-y* axis and unsymmetrical about the *x-x* axis.

For members similar to what is shown in Figure 5-8(a), it is possible to camber the member with bending about the *x-x* axis, but it may not be possible to fit the member in a cold-cambering machine due to the nonuniform cross section. The fabricator would likely be required to use heat to camber a member such as this, which can be costly relative to installing a stiffer member.

For members similar to what is shown in Figure 5-8(b), attempting to camber about the *x-x* axis is likely to result in warping of the cross-sectional shape. Therefore, it is not advisable to try to impose camber on members that are not symmetric about the *y-y* axis.

Fig. 5-7. Façade support attached to spandrel members.



(a) Unsymmetrical about the x-x *axis* *(b) Unsymmetrical about the* y-y *axis*

Fig. 5-8. Examples of nonuniform cross sections.

Appendix A
Floor Levelness

A.1 DEFINING LEVEL

Elevated structural floors are never perfectly level regardless of whether they are precast concrete, flat slab concrete (reinforced or post tensioned), or composite steel construction. Most engineers realize that floors are cast unevenly, beams and girders deflect, columns and concrete cores shorten, foundations may settle, and there are construction tolerances. The real question is what is acceptable and how do you achieve and measure it? Levelness is a serviceability consideration that will vary with the use and finish of the space. It is almost never a structural safety issue, except possibly in a case such as ponding, where so much additional concrete has been placed to level the floor that the live load capacity of the structure has been reduced.

Most floors are designed and constructed to an acceptable levelness using standard design and construction industry practices. The "problem," when it occurs, is the result of divergent expectations of the owner, architect, structural engineer, and the contractors involved. It starts with a failure to understand what the levelness requirements should be and, if there are special requirements, where they should apply. Next there is a need to clearly convey what the expected performance of the structural system will be in order for the contractor to properly plan the work. Construction procedures must be developed that can be used to achieve the required levelness. This includes a plan to take measurements and make adjustments that might be needed to achieve the required levelness.

An early report entitled "Design & Construction Issues for Achieving Floors of Acceptable Flatness and Levelness" by the Structural Engineers Association of Texas (SEAoT, 1994) provides a summary of the many issues involved. As the title indicates, this involves design and construction, as well as defining what is acceptable.

There is a difference between level and levelness. *Level* implies a surface with a tolerance to a horizontal plane. *Levelness* implies a tolerance to a plane surface that may or may not be level. Flatness characterizes the degree to which a surface conforms to a plane that may or may not be level. Flatness can be thought of as smoothness and is primarily controlled by the contractor's finishing operations. The key to this is to re-straighten the surface with a bull float or the use of a riding trowel with float dishes. Levelness—which is an average change in elevation over a longer distance—depends on the strike-off accuracy, as defined in the following, and the deflection of the structure.

ACI 117, *Specifications for Tolerances for Concrete Construction and Material* (ACI, 2010), has provisions for measuring flatness and levelness along with recommended floor flatness, F_F, and floor levelness, F_L, values. The levelness standards, F_L, are applicable only to slabs on grade or shored slabs and not to unshored suspended structural slabs. Some consultants specify these values for unshored floors anyway, contrary to the ACI 117 provisions, and this may lead to disputes. Special levelness requirements will increase the cost of the structure, and if they are not required for serviceability, they do not increase the value to the owner.

Levelness as stated previously depends on the strike-off accuracy when placing the concrete and the deflection of the structure. There are three primary methods of controlling the accuracy of the strike-off when placing concrete:

- The wet-screed guide uses a section of wet concrete that is struck-off to elevation.
- The dry screed uses a metal screed guide that is set to control the strike-off.
- The laser-screed method uses a machine that adjusts to maintain a constant elevation from a target.

ACI 302.1R, *Guide for Concrete Floor and Slab Construction* (ACI, 2015), addresses the methods used for concrete strike-off. Section 10.3.2 states that "wet-screed guides should be used only for surfaces where floor levelness is not critical." It goes on to state, "For suspended-slab construction the desirability of using dry-screed guides on both sides of each placement strip is diminished by the damage done when the contractor retrieves the guide system." For this reason, as shown in Figure A-1, a combination of dry-screed and wet-screed guide techniques is typically employed on suspended slabs. The wet-screed guide in this case is the top of the previously placed slab. ACI 302.1R is a guide, not a specification, and these are only recommendations; the wet-screeding method is still used by some contractors as noted in *Concrete Construction Magazine* (Leiferman, 2009).

There are also self-propelled screeding machines, such as those shown in Figure A-2, that can be laser guided to elevation or adapted to gage off the steel deck. ACI 302.1R, Section 3.3.5, cautions that when using a laser or similar instrument for the purpose of establishing uniform elevation for strike-off, the frame must be preloaded to allow deflection to take place before strike-off.

A.2 UTILIZING CAMBER WITH FLOOR LEVELNESS

There are two different methods of construction that are typically used to place the structural floor slab for unshored construction—placing the concrete to a constant thickness or placing the concrete to a constant elevation, as shown in Figure A-3. Both approaches are used throughout the industry and are structurally acceptable. Beam camber is commonly used with both methods to optimize the beam size while still conforming to the project-specific deflection requirements. By cambering the beams to offset a portion of the structure dead load (slab self-weight, deck weight, and beam weight), the theoretical floor elevation will be approximately level once concrete placement is complete. The balance of the allowable deflection is then evaluated against the post-composite loads, resulting in a more efficient steel system design than would be achievable without the use of camber.

The design variables listed in Chapter 4 will affect the actual beam deflection relative to the anticipated deflection used to determine the amount of camber. The variables that are expected to have the most impact on the beam performance are the connection restraint, variation in deck reaction based on span condition, fabrication tolerance, and possible span length reduction at column lines.

Fig. A-1. Dry screed guide and wet screed guide.

Fig. A-2. Laser screed.

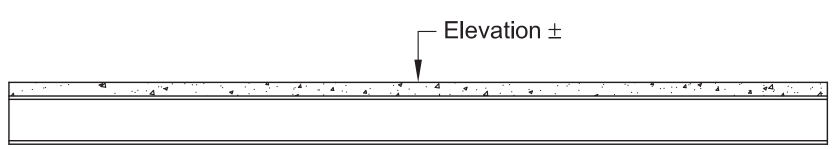
Beam camber where concrete is specified to be placed to a constant thickness are designed to achieve a uniform top of slab that is approximately level under the precomposite load, as shown in Figure A-3(a). The design variables can cause the in-place beam deflection to vary from the design, and as a result, the finished top surface of the floor surface will also vary. Any variation that does occur should be acceptable for standard interior finishes. When using this method of concrete placement, the dry-screed guides are set to the specified slab thickness so regardless of any difference in actual beam deflection and the camber provided, the slab will have the specified thickness. If a field survey of the first slabs placed shows significant variation in deflection from the specified camber, it is possible when using adjustable dry-screed guides to compensate for this variation in similar areas when placing the concrete.

Alternatively, designers may use the second method, shown in Figure A-3(b), and specify that concrete be placed to a constant elevation. The goal is to place a level slab surface even if the actual deflections of the beams vary from the theoretical calculated deflections. There are several key design and construction issues that should be addressed when using this approach to ensure a level slab.

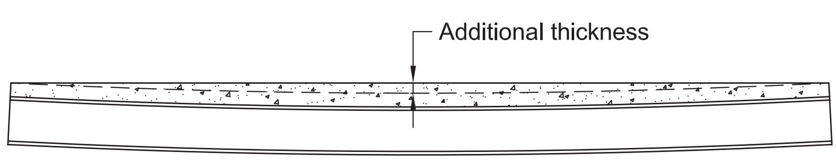
It is important when using the constant elevation method of concrete placement that the specified camber be less than the actual in-place deflection of the beams. If the in-place beams still have an upward curvature after concrete placement, the slab thickness at midspan may be less than specified. For many structures the specified slab thickness is the thickness needed to achieve the required fire rating. If this thickness is not achieved, costly fireproofing may have to be added to the underside of the deck to achieve the required fire rating. In extreme cases, the strength of the composite floor system might be reduced. There can be another problem with thin slabs when the steel headed studs project above the top of the finished floor. This problem tends to occur because the dead load deflection of beams with this type of slab is relatively small, and it is common practice to specify a camber that is a standard percentage of calculated dead-load deflection. Chapter 4 discusses how the deflection will vary with the type of slab and span and thus using the same set percentage of camber for all designs can result in substantial variation in the net midspan deflection. A more uniform method is to specify a camber that will provide the net beam deflection at midspan under precomposite dead load that the designer wants to achieve.

When specifying beam cambers less than the calculated precomposite dead load deflections, the deflection calculation should include the effect of the additional concrete required to achieve floor levelness. Because the deflection calculation is an inexact process, engineering judgment is often used to approximate this effect. The need for this additional concrete should be communicated to the contractor in the contract documents to allow for the added concrete in the bid.

When the constant elevation method for concrete placing is specified, it is important as noted in ACI 302.1R that beams are fully deflected before the concrete in the area is screeded. This will require the entire bay plus the adjacent bay be loaded to ensure both the beams and the supporting girder are fully deflected. Figure A-4 shows the floor being preloaded while laser screeding takes place in the background. While there have been anecdotal reports that the deflection may be time dependent, screeding usually starts as soon as the area is properly loaded. The screed guide or laser screed control is set to an elevation that will provide the specified minimum slab thickness over the high point of the deflected steel that is typically adjacent to a column. It is good practice to check slab depths at key points to verify that the beams have deflected approximately as expected per plan.



(a) Place concrete to a constant thickness



(b) Place concrete to a constant elevation

Fig. A-3. Concrete placement options.

When specifying camber for either method, it is important when calculating the theoretical deflection of beams at the column lines to consider how the effect of increased connection restraint, possible deck span effect, and possible span length reduction discussed in Chapter 4 may combine to reduce deflection. The deflection at the column line may vary from 75 to 90% of the typical interior beam deflection, depending on the magnitude and presence of each of these variables. Even though the member size specified is the same as the interior beam, engineering judgment should be used to reduce the specified camber at column lines.

The constant slab thickness method is typically the most economical method from a concrete cost standpoint. It can also be designed to provide some positive camber to offset the concrete shrinkage effect that will occur. The constant elevation method theoretically should provide a more level floor, especially when struck-off with special screed guides such as a laser-guided screed. The constant elevation method does, however, require additional concrete and special placement procedures. Either method can produce a serviceable floor.

A.3 LEVEL FLOOR CONSTRUCTION

The traditional strict division of responsibilities where the structural engineer is responsible for design and the contractor is responsible for the means and methods of constructing the work can hinder the process of constructing a level floor. The engineer is the party that sizes the members and specifies the camber along with determining the levelness requirements for the floor. The contractor controls the placement and finish of the concrete, but the supporting structure will vary in elevation and move as the concrete is placed. The contractor needs to know what the anticipated deflection will be and how his proposed method of placement might affect the levelness specified. Special conditions, such as floor beams adjacent to moment frames or chevron braced bays, need to be noted. As construction progresses, both parties need to know how the structure is performing and jointly determine if changes in concrete placement might be needed to achieve the required levelness.

This exchange of information can be facilitated by having a preconstruction meeting prior to concrete placement where the important design information and the planned concrete placement method can be discussed. A survey of the elevation of the in-place steel will help with this planning. The survey is not to verify the fabricated camber, which must be verified in the shop in accordance with the AISC *Code of Standard Practice* (AISC, 2016a), but to compare the in-place camber of each beam and their support elevations to the concrete placement plan. This can be used to help anticipate how much deflection will occur when the concrete is placed. After the initial concrete placement, another survey can be made to determine if any changes in the concrete placement procedure should be made in the following areas based on the actual deflections of the structure.

Where exceptionally level floors are needed, the engineer

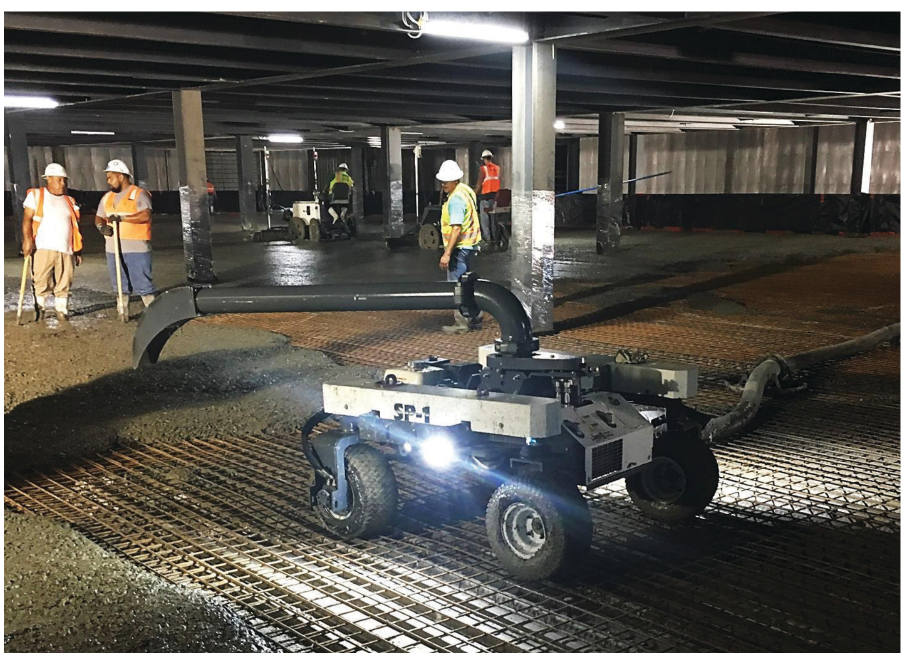


Fig. A-4. Floor preloading.

might consider increasing the size and/or depth of the typical floor beam to reduce the camber required. The reduction in the amount of camber required will make it somewhat easier to adjust for deflection when placing concrete. For special cases, it may be necessary to specify a topping slab; however, this will not only increase the cost but will also impact the schedule.

A.4 STRUCTURAL STEEL FRAME TOLERANCES

When specifying floor levelness, the designer should be aware that structural steel frame tolerances may affect the elevation of the floor members. This is typically not a problem for most low-rise structures. Structural steel frame tolerances are established in the AISC *Code of Standard Practice*. The horizontal floor beams that connect to a column are not set to a specific elevation but are set a specified distance from the top of the column piece to which they frame with a tolerance of $+\frac{3}{16}$ in. to $-\frac{5}{16}$ in. The elevation at the top of the column may vary from theoretical plan elevation due to fabrication and erection tolerances along with foundation settlement and the effect of column shortening. The top of floor elevations will then vary from the actual beam elevations, which will vary based on the previously mentioned tolerances. The actual in-place beam elevations are typically acceptable for serviceable floor construction. For low-rise structures less than 10 stories, these elevation differences will probably not require column adjustments. There may be special framing conditions such as complete-joint-penetration (CJP) groove welded moment frames or lightly loaded mullion columns where some adjustment should be made. AISC Design Guide 21, *Welded Connections—A Primer for Engineers* (Miller, 2018), discusses weld shrinkage and how to compensate for it when detailing CJP groove welded joints.

Design practice for concrete core buildings up to 30 stories has been to neglect column shortening because it tends to compensate for the concrete core shrinkage. Some projects now set core elevations to compensate for shrinkage, and it is important to match this with increased column lengths to offset the column shortening effects. Where dead load stresses in columns, such as moment frames or mullion columns, are low, the column shortening effects will be reduced. The design drawings should provide special instructions for detailing the lengths of all of these members. The tops of all columns should be surveyed when erected to determine actual elevations and to properly plan the work. For high-rise buildings, provisions can be made to adjust column elevations at splices by shimming or adjusting the root gap as permitted by AWS D1.1/ D1.1M (AWS, 2020).

Appendix B
Rules of Thumb

Rules of thumb are principles with broad application that are not intended to be strictly accurate or reliable for every situation. These are quick guidelines to be applied generally, but with due consideration, may be violated. Each of these items has been discussed in greater detail in the preceding sections, providing some additional insight as to why these rules of thumb exist and when they may not be applicable. They are summarized here as a convenience.

1. **Show required camber for each member on the framing plans or in the model.**
   This will clearly show any variation in cambers at special framing conditions.

2. **Reduce camber for members framing into columns.**
   Consider the effect of reduced clear span, increased connection restraint, and reduced concrete weight associated with the deck-span end reaction when specifying camber at column lines.

3. **Require that camber be verified in the fabrication shop by quality control and/or quality assurance.**
   This is the safest, most reliable, and most cost effective way to inspect camber.

4. **Avoid putting holes in the top flange near the middle third segment of the beam.**
   The cambering process will yield the beam and possibly cause net section rupture at the holes.

5. **Do not specify camber less than ¾ in.**
   This slight variation in deflection should not have a significant effect on overall floor levelness.

6. **Do not camber members with spans less than 25 ft.**
   Beams less than 25 ft may require special procedures due to machine limitations. It may be more cost efficient to use a larger beam that eliminates the need for camber.

7. **Do not camber members with webs less than ¼ in. thick.**
   Thin webs tend to cripple when the beam flanges are loaded to yield.

8. **Do not camber members with moment connections, bracing connections, or special connections for torsional restraint.**
   Camber will make connection fit-up difficult. Use a larger member to eliminate the need for camber if necessary.

9. **Do not camber members with nonuniform cross sections.**
   These members tend to twist when strained to yield without special procedures.

10. **Do not camber crane beams or crane girders.**
    These members should be sized with adequate stiffness according to crane design requirements.

11. **Do not camber spandrel members that support brittle façade materials.**
    These members should be sized for stiffness rather than using camber to compensate for dead load deflections.

12. **Do not camber cantilever beams.**
    Provide adjustable connections and preset tip elevations using the AISC *Code of Standard Practice* recommendations instead of camber.

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