Technical Topics

One of the most effective ways to address the cost of a steel structure is to consider the relative costs of design decisions early in the process—during the conceptual and preliminary design phases. The following articles highlight the trade-offs of several design decisions. And don't forget: When in doubt, consult your local fabricator!

Floor Framing Considerations By Jason R. Ericksen, S.E.

When designing steel floor framing, the major considerations are the level of composite action, whether or not to camber, the bay dimensions, the beam spacing, and the depth of the floor framing.

To Camber or Not to Camber

Cambering is done to counteract the deflections of beams due to the weight of wet concrete during construction. The alternative is to use more steel material to reduce these deflections. The fabrication and erection cost of each beam will not change with moderate changes in the beam weight. Therefore, compare the cost of cambering to the cost of the material increase.

Depending on the architectural and mechanical details, increased beam depth can increase floor-to-floor height and building height. This could lead to increase in cladding costs and mechanical systems costs. In general, the cost to camber a beam is $50–$65 for a range of camber from ¾" to 2"–2½". Steel material is running about $0.30/lb. So, the current approximate cost to camber a beam is worth 167 lb to 217 lb of steel. There are other costs that are affected by changing the beam size that must be considered, such as the cost of shear studs.

If you decide to camber, remember to allow short slotted holes in the end connections of the beams to allow for the geometry of the curved member. Don't over specify camber. If not enough of the camber is taken out while the concrete is being poured, the shear studs may protrude. Camber two-thirds to three-quarters of the calculated dead load for members of length 20' to 40' respectively to account for connection end restraint. Cambering members more than 2" to 2½", depending on the machine, will require repositioning of the beam and will increase the cost of the camber by about 50%.

Composite Decisions

In many situations, composite action can significantly reduce the weight of the steel framing. The cost of an installed stud can vary greatly across the country, with an average of about $2.50 per stud. With steel at $0.30/lb, each stud is worth about 8 lb of steel. The steel weight savings using partial composite action over 50% to 75% is often overcome by the cost of the studs. Eliminating camber by using a heavier section, as discussed previously, can also have the economic benefit of reducing the required number of studs.

> "Consider non-composite construction when there are few members that benefit from composite action...the cost per stud could increase drastically without the economy of having plenty of studs to install."

Consider non-composite construction when there are few members that benefit from composite action on the job. There are costs to mobilize an installation crew, and the cost per stud could increase drastically without the economy of having plenty of studs to install.

Sizing Matters

Along with deciding the best way to design each member, the geometry of the floor system must also be investigated. This includes finding optimal bay dimensions and beam spacing. John Ruddy (*Engineering Journal*, Third Quarter, 1983) suggested that using a bay length of 1.25 to 1.5 times the width, a bay area of about 1000 sq. ft, and filler beams spanning the long direction combine to maintain economical framing. Typically, efficient beam spacing ranges from 8' to 13'.

Larger bays and greater spacing mean deeper and heavier beams, but there will also be fewer pieces of steel and connections to detail, fabricate, and erect, saving time and money. In addition, there will be fewer, albeit heavier, loaded columns and foundations. The cost of additional building height, due to increased structural depth, also needs to be considered.

Close coordination with architectural ceiling requirements such as placing beams at wall partitions can reduce the impact of a deeper structure. Coordination with the mechanical systems can also keep the floor heights down by running ductwork through openings in the beam webs instead of below the structure.

The Steel Tool® Parametric Bay Studies was created to help engineers look at the relative costs of floor framing systems. By varying the bay geometry and beam spacing, the tool can help the user decide to use composite or non-composite design, when to camber, and when to increase the beam size to reduce the number of studs. It can also help determine optimal beam spacing and optimal bay geometry by combining the results from the tool with costs for foundations, metal deck, and increases in floor height. Additionally, it can show how much a decrease in structure depth will cost in terms of the floor framing. ★

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Column Stiffening Considerations

Excerpted from AISC's *Design Guide 13*

According to AISC's *Design Guide 13, Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications*, eliminating transverse stiffeners and web doubler plates can result in significant cost savings. Designers should consider alternatives to these applications for strengthening columns whenever possible.

One option, according to the guide, is to specify column material with 50 ksi yield strength, like ASTM A992 or A572 grade 50 steel. "The increased minimum yield strength will increase the design strength of the column, yet there will be little or no impact on the material cost," the guide states.

The guide also recommends using a different column section that has a thicker flange and/or web, if appropriate. Increased material costs would be offset by the amount of labor saved.

A deeper cross-section for the beam connected to the column should also be considered. "Increasing the depth of the beam decreases the flange force delivered due to the increase in moment arm between the flange-force couple," the guide explains. If a lighter, deeper shape is suitable, material and labor costs would be decreased. The guide notes, however, that when the moment connection is designed to develop the strength of the beam, this suggestion is potentially punitive.

Finally, the guide recommends increasing moment-resisting connections to avoid the use of stiffeners or doubler plates. "Increase the number of moment-resisting connections and/or frames to reduce the magnitude of the moment delivered to a given connection to a level that is within the local design strength of the column section," the guide says.

In some cases, stiffeners and doubler plates cannot be avoided. The guide has several suggestions to help reduce the cost impact, including:

1. Design column stiffening in response to the actual moments and resulting flange forces rather than the full flexural strength of the cross-section. When the EOR delegates the determination of the column stiffening requirements, the design forces and moments should also be provided.
2. If designing in allowable stress design, take advantage of the allowable stress increase in wind-load applications (load combinations in LRFD inherently account for such concurrent occurrence of transient loads).
3. Properly address reduced design strength at column-end applications. The typical beam depth is usually such that the reduced design strength provisions for column-end applications apply only at the nearer flange force.
4. Increase the number of moment-resisting connections and/or frames to reduce the magnitude of the moment delivered to a given connection to a level that allows a more economical stiffening detail.
5. Give preference to the use of fillet welds instead of groove welds when their strength is adequate and the application is appropriate. This is particularly true for the welds connecting transverse stiffeners to the column.
6. When possible, use a partial-depth transverse stiffener, which is more economical than a full-depth transverse stiffener because it does not need to be fitted between the column flanges. Select the partial-depth transverse stiffener length to minimize the required fillet-weld size for the transverse-stiffener-to-column-web weld.

These tips, and six more, can be found in AISC's *Design Guide 13*, available free online to AISC members and ePubs subscribers at **www.aisc.org/ePubs**. ★

All About Bolts

By David I. Ruby, P.E., S.E.

Why spend the time choosing between snug-tightened, pretensioned, and slip-critical joints? Simply put, snug-tightened joints are the most economical bolted joints and should be specified wherever possible. Slip-critical joints are by far the most costly joints, and should be specified only when the unique qualities of slip-critical joints are required for proper joint performance.

The table below (using one fabricator's estimate of labor costs) shows clearly that snug-tightened bolts are cost-effective.

Estimating Bolting Costs

Consider a 59 kip factored load using ASTM A325 high-strength bolts. The cost estimates include one fabricator's estimate of the associated labor costs:

			Cost	Cost Factor
Slip Critical (N or X)	6	bolts @ 10.4 kips/bolt = 62.4 kips	$66.00	3.1
Pretensioned (N)	4	bolts @ 15.9 kips/bolt = 63.6 kips	$34.00	1.6
Pretensioned (X)	3	bolts @ 19.9 kips/bolt = 59.7 kips	$25.50	1.2
Snug-tightened (N)	4	bolts @ 15.9 kips/bolt = 63.6 kips	$28.00	1.3
Snug-tightened (X)	3	bolts @ 19.9 kips/bolt = 59.7 kips	$21.00	1.0

The Structural Engineer of Record primarily is concerned with the structural adequacy of the facility, while attempting to design the lightest structure—which is often erroneously assumed to correspond to the least cost. Actually, the cost is more influenced by labor in the detail connections, the shop and field hours required for material preparation, punching, drilling, cleaning, painting, masking, inspection, installation, and testing of the high-strength bolts.

Slip-critical joints cost appreciably more because of the associated faying surface preparation requirements (see Section 3.2.2 of the RCSC *Specification*), which can include:

1. Removal of loose mill scale (achieved by power-brush cleaning or brush blasting)
2. Removal of burrs on punched or drilled holes (achieved with hand or power grinding)
3. Special coating options:
   - free of coatings, including over-spray (which requires masking of surfaces prior to painting)
   - special paint systems that are rated for slip-resistance (consult a steel fabricator).

Slip-critical joints also require more design time because all the limit states for the bearing condition must be checked in addition to that for slip-resistance. Even though a slip-critical joint has a calculable frictional resistance, the joint can slip into bearing and must have adequate bearing strength should that happen (read more about this in the Commentary to Section 5.4 of the RCSC Specification).

Whenever possible, specify snug-tightened joints. If snug-tightened joints are not permitted, and slip-resistance is not required, specify pretensioned joints. Simply using the proper joint type can save a lot of money on your project. ★

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Relative Economies of CJP vs. PJP Groove Welds

By Duane K. Miller, P.E. and Thomas J. Schlafly

The choice between partial joint penetration (PJP) and complete joint penetration (CJP) groove welds depends on application and economy. There are some cases where only CJP will do the job. **The importance of properly addressing this issue cannot be overemphasized: There is no point in comparing the relative economics of a PJP versus a CJP if a CJP is required for the application.** Thus, some discussion regarding the selection of required weld details is in order.

Basic Differences

CJP welds are designed with the use of a matching strength filler metal and a prescribed set of provisions. When these provisions are met, codes (such as the AISC *Specification* and AWS D1.1 *Structural Welding Code*) permit the weld available strength to be taken as the base metal available strength. PJP welds need not be made with matching filler metal, nor do they have the effective weld size equal to the adjacent base metal area. In fact, many prequalified PJP groove welds have an effective size ⅛" less than the bevel.

Further, to preclude melt-through, PJP groove welds have a minimum root face dimension that subtracts from the amount of cross section that may be used to supply fused metal. Therefore, PJP welds in some joints, such as butt joints, cannot be made with an available strength equal to that of the base metal. Tee joints, on the other hand, can be reinforced with fillet welds and can be designed to meet the base metal strength. In some cases, this is an effective method of achieving a "full strength" connection, but without resorting to CJP groove welds.

Load Considerations

The nature of the applied loads may dictate the use of CJP groove welds. For example, cyclically loaded joints subject to fatigue are designed for anticipated stress ranges. Some fatigue details require the use of CJP welds. In seismic design, some members are detailed to anticipate and accommodate significant inelastic strains in the base metal that subject the weld to strains above yield. Therefore, some qualified or prequalified seismic joints require the use of CJP welds. There are occasions where bending puts tension on one side of the joint. This does not preclude the use of PJP groove welds, but it may be wise to eliminate single bevel PJP groove welds where there is tension across the root of the weld.

Weld Symbols

To specify a CJP groove weld, all that is required is to show 'CJP' in the tail of the weld symbol. It should be noted, however, that when this is done, other alternatives that may be acceptable in some situations (such as the aforementioned example of PJPs with the addition of fillet welds) are precluded.

When only the strength of the base metal must be developed (e.g., both CJPs, PJPs, and fillets are acceptable) the design drawing need only show a groove weld symbol with no dimension for the weld size. That symbol indicates that the fabricator can use any combination of fillets and PJP welds or CJP groove welds to develop the required strength. Of course, if the base metal strength does not need to be equaled, showing the required load or weld size will permit significant economy and possibly reduce distortion and residual stress issues.

Cost Concerns

The economical comparison of PJP and CJP joints depends on the factors behind the selection of the weld.

If a PJP weld is sufficient for the application, the alternate use of a prequalified CJP groove weld requires the use of steel backing or backgouging and rewelding. The backgouge and reweld may require the equivalent time as would be required to make three or four extra weld passes. Steel backing requires the extra time to fit and tack the bar, and the joint geometry entails a root opening that increases the cross sectional area of the weld.

In a 1.5" thick plate, a PJP groove weld with the largest prequalified bevels might require 1.60 lbs of weld per foot, while a CJP weld might have 3.5 lbs/ft including 1.0 lbs for the backgouge and reweld. Where a PJP less than the maximum size can be used, the savings is proportional to the difference of the weld areas or the difference of the square of the throat of the two welds being compared.

Therefore, if a ¾" PJP will suffice, it can save on the order of $1- 0.75^2/1.5^2 = 75\%$. If backgouging and rewelding is not done, steel backing bars are required. Tacking the backing in place and assembling the joint with CJP groove welds is more complicated than assembling members with simpler PJPs.

It is nearly universally true that a PJP groove weld will always be lower in cost than a CJP groove weld. Only one exception comes to mind, as follows: If electroslag (ESW) or electrogas (EGW) welding processes are used (both of which are almost always used to make CJP groove welds), it is possible that a CJP groove weld may be lower in cost to make than a PJP. Outside of this limited exception, it is likely that the PJP will cost less than the CJP.

Additionally, other advantages may be seen. Lamellar tearing is driven primarily by the shrinkage strains induced by welding. Reduction in the volume of welding has always been one means of mitigation of lamellar tearing tendencies–yielding advantages for the PJP detail.

Inspection

A discussion of CJPs versus PJPs would be incomplete if the topic of "inspection" was not addressed. Often, complete joint penetration groove welds are specified, and justified, as follows: "If I don't specify CJP, I don't get any inspection."

This thinking is flawed on multiple levels. First, **all** welds performed in accordance to AWS D1.1 are required to receive visual inspection, which when properly performed involves a whole host of inspections performed before, during, and after welding. The intent, of course, of the "justification" is that nondestructive testing is not possible, but this is not totally correct either. While radiographic (RT) and ultrasonic (UT) testing are difficult or impossible to perform on PJP groove welds, dye penetrate (PT) and magnetic particle (MT) inspection are possible. And, finally, it should be noted that, unless RT or UT of CJP groove welds is specified, even nondestructive testing (NDT) is not automatic.

Conclusion

In conclusion, if either CJP or PJP groove welds are acceptable, PJP details will, in nearly every case, reveal significant savings. An example of a 3:1 cost savings was shown. While many factors are involved, a rough rule-of-thumb to estimate savings would be to take the square of the resultant weld throats and compare them. For the example cited, using this rule-of-thumb, the ratio would be $(0.75)^2/(1.5)^2$ or 0.34 (e.g., 75% savings).

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Seismic Design: $R = 3$ or $R > 3$

By Charles J. Carter, PE., S.E.

Current International Building Code (IBC) requirements for seismic design are based upon a system of Seismic Design Categories (SDC), lettered A through F. In SDCs D, E, and F, the designer is required to use a seismic response modification factor $R > 3$ and meet the corresponding detailing requirements in the AISC *Seismic Provisions for Structural Steel Buildings*. In SDCs A, B, and C, however, the seismic hazards are generally lower. The designer can choose between $R = 3$ with detailing requirements of normal ductility (those in the AISC *Specification*) and $R > 3$ with the higher ductility AISC *Seismic Provisions* detailing requirements.

When permitted to choose, should a designer use $R = 3$ or $R > 3$? The use of $R > 3$ results in a lower base shear, which misleads many into thinking that the system will cost less because lower forces mean lighter framing. But this is rarely the case. The use of $R > 3$ invokes the detailing requirements in the AISC *Seismic Provisions*, which include member requirements and connection requirements. Braces, beams, and/or columns must satisfy minimum b/t ratios and member force requirements. The connections of these members must often develop the full strength of one of the members. As a result, the lateral framing systems for $R > 3$ will almost always be heavier than lateral framing systems for $R = 3$, even though the seismic base shear is smaller.

> "The use of R > 3 results in lower base shear, which misleads many into thinking that the system will cost less because lower forces mean lighter framing."

Even if they were of similar weight, the connection costs in $R > 3$ systems are much higher than in $R = 3$ systems.

There are three factors that influence the SDC classification: the location on the map (acceleration potential), the importance factor for the building, and the soil conditions at the site. Sometimes the site is such that the standard map-based SDC classification shifts to SDC D. It is often the case that a special soils investigation (for dynamic soils properties, not just for foundations work) will allow the site to be classified as SDC C instead. If this is the case, the economy of the framing will often save far more than the soils report will cost.

Special note: Often in the past, and sometimes with explicit language in the building code allowing it, an engineer has used $R > 3$ in an application in SDC A, B, or C with no special detailing because "wind controlled". This was never technically correct because the seismic response modification used presumes the ductility necessary to achieve it is present. Current IBC requirements eliminate this loophole of the past—if you use $R > 3$, you must detail the system accordingly. ★

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